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VOL. 74 JANUARY, 1948 NO. 1

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AND

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

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A SYMPOSIUM

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INTRODUCTION

BY WILLIAM H. SMITH,[1] M. ASCE

The Bureau of Yards and Docks, United States Navy Department, has always been deeply involved in the problems of heavy earthwork. Much of the Navy's shore establishment is located on existing harbors and waterways, where the engineer must accept foundation and subsoil conditions as he finds them. Almost any harbor improvement project involves filling out beyond the natural shore line, building a quay wall or bulkhead, and dredging outside it. In many cases the engineering problems of the Navy are rendered more difficult because of the heavy surcharges to be provided for. The weights of guns, barbettes, turrets, armor, and other components of capital ships, the generally heavy character of equipment, and the tendency to do more and more prefabrication and preassembly—all lead to heavier types of shore construction.

On the other hand, rising costs and shortages of materials lead to progressive development of designs to utilize modern theories, materials, and methods to the best possible advantage. The days of massive, granite-faced gravity walls, for example, seem to have passed. These developments impose greater and greater responsibilities on engineers to accomplish economical results without reducing the factor of safety inherent in their structures.

The best way to improve engineering design is to improve knowledge of the forces to be resisted and of the mechanics of the structures in resisting them, so that every pound of material is working at the fullest possible efficiency and yet is never overstressed. The test program undertaken in 1943 at Princeton University (Princeton, N. J.) under the sponsorship of the Bureau of Yards and Docks has this specific objective in view. By careful laboratory studies of lateral earth pressures, and, particularly, studies of methods to reduce the lateral fluid pressure of hydraulically placed backfills, engineers can insure against failures of bulkheads and can arrive at more economical designs that will be safe and reliable. By further study engineers can learn even more about the behavior of flexible bulkheads, and the effect of bulkhead displacement on soil pressures, so that by modifying designs or methods of construction or both, they can build better bulkheads at lesser cost.

Briefly, the broad topics studied under this program can be divided into four main categories. First, investigation was launched into the determination of lateral pressures exerted on flexible steel bulkheads by backfills of clay, a sand-clay mixture, and a clean sand. This phase of the problem was important in order to obtain the basic information as to the lateral pressures to be expected with backfills of these types. The second phase of the problem was to determine methods for reducing these pressures. Tests were conducted to determine the effectiveness of sand dikes, placed on a normal one-on-two slope behind the bulkhead, in reducing lateral pressures. The effectiveness of the sand dike and of other methods attempted will be covered in the first Symposium paper. As the third phase of the investigation, and as a corollary to the sand-dike experiment, tests were made to determine the effectiveness of a sand

[1] Rear Admiral, CEC, U. S. Navy, Washington, D. C.

blanket of varying widths placed back of the bulkhead in reducing lateral pressures. A fourth investigation was launched into the development of a method of accelerated consolidation of clay backfills designed to reduce quickly the excess pore pressures in the backfill and, correspondingly, to reduce the lateral pressure on the bulkhead. Finally, as a result of these studies, data of basic importance have been obtained for the analysis of the phenomena related to the problems of lateral pressures of soils, and many avenues of approach for further research and study in this field have been revealed. The present status of the tests, the preliminary findings and conclusions, and the contemplated future course of action on the completion of the test program are fully discussed in the papers.

Several aspects of the Symposium deserve emphasis:

(1) It is not a final statement of results. It is rather a progress report on what has been accomplished, a discussion of the tentative conclusions and interpretations of the test results, and an indication of the direction that further tests will take.

(2) Although there have been free and intensive discussions among Professor Tschebotarioff, his associates at Princeton, and the designing engineers of the Bureau of Yards and Docks, the Navy has not attempted to force any unanimity of conclusions or opinions. It is only by the application of basic principles of free speech—by the full and free presentation of divergent points of view—that a meeting of minds and a constructive, intelligent, and sound set of principles can be obtained.

(3) It is neither the intention nor the hope of the sponsors of this investigation to resolve, once and for all, the many open questions in the field of soil mechanics. In one who does not attempt to qualify as an expert specialist in this science, there frequently arises a feeling of confusion and futility when one tries to reconcile the deep and apparently fundamental disagreements between different authorities on the interpretation and application of the theories of soil mechanics.

The general practitioner in civil engineering must make allowances for the relative youth of this branch of engineering as a science. He must recognize the complexity and variability of the materials with which these experts deal, and must have patience with them in their efforts to bring law and order into an involved and intractable situation.

Medical doctors have been struggling for years to solve the mystery of the common cold. Perhaps, in comparison, the engineering profession should applaud soils engineers for the great progress they have made in clarifying and delineating the behavior of mother earth, rather than look askance at their achievements because they are occasionally clouded by controversy. These active differences must be recognized rather, as proof in themselves of the virility, the vigor, and the dynamic character of this young science.

NOTATION

The relatively few letter symbols in this Symposium conform essentially to "Soil Mechanics Nomenclature," ASCE *Manual of Engineering Practice No. 22*. Discussers are requested to adapt their symbols to this list:

A = reaction at level A;
A' = the slope of the semilogarithmic plot of a void-ratio curve (Eq. 30); $A = A'/2.306$;
B = reaction at level B; also B = a constant in Eq. 30;
C = reaction at level C;
C' = a constant defined by Eq. 9;
c = cohesion per unit area:
D = diameter; average diameter of a pile;
E = modulus of elasticity in bending and tension; E_G = modulus of elasticity in shear;
e = void ratio:
- e_{cr} = critical void ratio;
- e_s = void ratio at the surface, $h = 0$;
- e_t = void ratio corresponding to $p_s = 0.1$ ton per sq ft;

$\mathbf{e}$ = base of Naperian logarithms;
G = specific gravity of solids;
H = height of bulkhead:
- H'_c = critical depth, or the depth at which a vertical cut will stand unsupported;
- H_g = depth of ground water level below the ground surface;

h = vertical distance from the top of the bulkhead to any point along its vertical height:
- h_i = the depth where $(\gamma_a - \gamma_w)h_i$ is 0.1 ton per sq ft;
- h_o = distance h to any point along the vertical height of a bulkhead;

I = rectangular moment of inertia;
I_p = plasticity index;
K = hydrostatic pressure ratio; ratio of horizontal pressure to vertical pressure; a consolidated pressure coefficient:
- K_A = ratio for the reaction A at the anchor level A;
- K_a = ratio for active pressure;
- K_B = ratio for the reaction B at the anchor level B;
- K_h = ratio of pressures at depth h;
- K_p = ratio for passive pressure;
- K_R = ratio for the total pressure R;
- K_s = ratio for the buoyed weight of solids;
- K_{s+w} = ratio for the nonbuoyed combined weight of soil and water;
- K_ϵ = ratio for the bending strain, ϵ_o;
- K_1 = active pressure coefficient for a triangular sand dike in Fig. 40;
- K_2 = active pressure coefficient for a level sandbank;

L = span length; vertical distance between the A-level and the B-level of the model bulkhead; L_p = length of pile;
M = bending moment;
m = a factor defined by Eq. 29;
N = a ratio of K-values; N_o = the value of N for any height h_o;
n = tangent of the angle of wall friction;
P = pressure force exerted on a bulkhead (pounds per linear foot):
- P_a = total active force exerted on a bulkhead (pounds per linear foot);
- P_{af} = total active force exerted on a bulkhead, considering wall friction;
- P_{ax} = total active force exerted on a bulkhead for a variable angle, X, of rupture plane;
- P_{cr} = critical load on a pile;
- P_p = total passive force exerted on a bulkhead (pounds per linear foot);
- P_1 = total horizontal pressure due to weight W_1 (Figs. 43 and 45);
- P_2 = total horizontal pressure due to weight W_2 (Figs. 43 and 45);
- P_3 = total force from fluid clay (Figs. 43 and 45);

p = unit pressure (pounds per unit area):
- p_a = unit active pressure;
- p_c = capillary pressure;
- p_d = difference in unit pressures;
- p_h = unit lateral pressures;
- p_n = normal pressure;
- p_s = consolidation pressure;
- p_1 = the major principal stress; unit pressure for a triangular dike;
- p_2 = the minor principal stress; unit pressure for liquid clay;
- p_3 = unit pressure for a triangular dike;
- p_4 = unit pressure for liquid clay;

Q = ratio of friction angles n/μ;
q = unit compressive strength of unconfined specimens; q_s = unit strength of specimens consolidated under pressure corresponding to effective weight of overburden;
R = resultant force; total lateral pressure; horizontal reaction; subscripts indicate point of application;
S = ratio of pressures, P_{af}/P_a;
s = spacing distance; as a subscript, s denotes "solids"
t = a numerical factor less than 1.0 (berm width); ratio of top width to height of trapezoidal dike;
V = shear force, in pounds;
v = shearing resistance, shear force per unit area;
W = total value of a distributed weight w:
- W_1 = buoyed weight of sand dike above rupture plane (Figs. 43 and 45);
- W_2 = buoyed weight of fluid clay above rupture plane (Figs. 43 and 45);

w = a variable distributed load or weight; as a subscript, w denotes "water";
X = angle between the assumed rupture plane and the horizontal;
z = distance from the top of the bulkhead to the point of maximum bending moment on the bulkhead; z_o = distance from the top of the bulkhead to the point of theoretical zero pressure on the upper part of the bulkhead;
α = tangent of the angle X:
α_a = tan X for active pressure;
α_p = tan X for passive pressure;
α_x = tan X;
β = tangent of the angle θ;
γ = weight per unit volume:
γ_a = unit weight of soil in air;
γ_e = effective unit weight of soil, as transmitted from grain to grain; equal to buoyed weight, when soil is submerged;
γ_w = unit weight of water;
γ_1 = buoyed weight per unit volume for sand;
γ_2 = buoyed weight per unit volume for clay;
ϵ = bending unit strain; ϵ_o = maximum strain of an equivalent beam freely supported at levels A and B;
μ = tangent of the angle ϕ;
ϕ = angle of internal friction; ϕ_q = slope of a line relating unconfined compressive strength to consolidation pressure; and
θ = surcharge angle (positive above level bank).

GENERAL ACKNOWLEDGMENT

The Bureau of Yards and Docks pays special tribute to the efforts that have been made, by Professor Tschebotarioff and his associates, in the preparation of this Symposium. Officers and designing engineers in the Department of Planning and Design, Bureau of Yards and Docks, also deserve hearty commendation. First, the duties imposed by World War II and subsequently the reduction of funds and personnel have added burdens on all of them. During the transition from a wartime to a peacetime footing it has not been possible to assign any Navy personnel to this task full time and all have had to do their share while handling many other assigned tasks.

LARGE-SCALE MODEL EARTH PRESSURE TESTS ON FLEXIBLE BULKHEADS

BY GREGORY P. TSCHEBOTARIOFF,[2] M. ASCE

SYNOPSIS

Large-scale model tests of the earth pressure on flexible bulkheads are described in this paper. The main design features of the testing installation at Princeton University (Princeton, N. J.) are outlined. A description is given of tests performed with submerged sand backfills, with clay and with sand-clay mixture backfills placed in a fluid condition, and with sand dikes and with vertical sand blankets of varying thickness backfilled with fluid clay.

Measurements are reported of the pressures exerted by the backfilled model flexible bulkhead, against its supports, of the bending strains and of the deflections of the bulkhead itself, and of pore pressures in the backfill during and after its consolidation. Results of laboratory soil classification tests and of shearing strength determinations are given. Critical comparisons are made between the model and the laboratory test results and the results of various known methods of analysis of the problems studied.

Findings are stated which should be considered when designing anchored flexible bulkheads. A method of approach to the problem of determining lateral earth pressures for bulkhead design is presented. Points requiring further clarification are outlined, as well as the tests that are in progress.

INTRODUCTION

In presenting this interpretation of findings at Princeton, the writer will need to make comparisons with previously accepted or published views. In the course of such comparisons he will refer to some relevant published statements by Karl Terzaghi, M. ASCE, not all of which have been fully substantiated by the tests described. No criticism of Professor Terzaghi is implied by such comparisons; these references serve as the most authoritative and most progressive available standard by which a comparison can be made of advances in the field of soil mechanics. The tests are concerned with problems about which little or no factual data were available. For that reason most previous recommendations concerning these particular problems necessarily were in the nature of guesses. The writer was impressed when he discovered how often Professor Terzaghi was right in extrapolating his recommendations beyond the realm of available data. The cases in which his judgment of incomplete or inadequate data was not vindicated only emphasize the importance of large-scale experimental research in the field of applied soil mechanics, a point often stressed by Professor Terzaghi himself.[3,4]

[2] Associate Prof., Civ. Eng., Princeton Univ., Princeton, N. J.

[3] "Anchored Bulkheads," by K. Terzaghi, *Proceedings*, Purdue Conference on Soil Mechanics and Its Applications, Lafayette, Ind., 1940, p. 270.

[4] "General Wedge Theory of Earth Pressure," by Karl Terzaghi, *Transactions*, ASCE, Vol. 106, 1941, p. 79.

Findings reported in this paper, as compared to previously held views, have shown the dangers of taking too much for granted where soils are concerned. Every effort is made, therefore, to indicate the possible limitations of established facts and to differentiate between these facts and the writer's personal interpretation of the significance of recorded data. Whenever an interpretation different from that of the writer has been advanced by one of the other Symposium authors, this fact will be mentioned.

Problems to which no answer has been obtained are stated and the measures taken for their solution are outlined. In this respect, the paper has the character of a progress report, since testing is continuing.

PART I. THE PROGRAM OF FIRST TEST SERIES

A. General Objectives of First Series of Eleven Tests

(1) The original objective set for the Princeton project in February, 1943, was to determine, by large-scale model tests, the quantity of sand that would have to be interposed between the sheet piling of an anchored flexible bulkhead and a hydraulic backfill with high clay content, in order to decrease, effectively, the fluid lateral pressures of the unconsolidated backfill against the sheet piling. Three tests of this type were performed in the succeeding four years.

The study of objective (1) involved combined operations with two limit types of soil backfills—with clean sands and with fluid clays. The installation that would have to be designed and built for this purpose could well be used for the determination of some practically important aspects of lateral pressures exerted by either type of backfill alone. The following two further major objectives were added to the program:

(2) Study of lateral pressures exerted against flexible bulkheads by fluid clay backfills and by fluid sand-clay mixtures before, during, and after their consolidation—the study to include procedures for accelerating consolidation; and

(3) Verification of the alleged[5] considerable decrease of bending moments in sheet piling resulting from the vertical arching of sand backfills between the anchor level and the soil below the level of the dredged line.

By the end of 1946 four tests each had been performed in the study of objectives (2) and (3), completing the test series with the first type of testing installation.

B. Selected Method of Approach to the Problem

The three objectives defined in Section A cover a wide range. The Bureau of Yards and Docks was naturally anxious to obtain quickly results which could be applied directly in the field. At the same time it was not possible to obtain results without exploring some of the most complicated theoretical aspects of the effective shearing strength of soils and other similar, only partly explored, branches of soil mechanics.

[5] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, pp. 217–219.

For the foregoing reasons no attempt was made to reproduce all possible combinations of relevant factors. Instead, tests were developed which would permit the estimation of the relative importance of factors likely to be relevant, and a study of the most important ones. To that end the program was frequently modified as the work progressed.

In the design of the installation emphasis was placed on tests with fluid clay backfills and special attention was paid to model similarity with steel sheet piling, in so far as the magnitude of bulkhead deflections was concerned.

C. Design Features of Testing Installation for First Series of Eleven Tests

The testing installation (see Fig. 1), under Palmer Stadium, at Princeton University, is believed to be the most extensive of its kind in existence. Nevertheless, as completed in 1944, with all measuring equipment and other accessories, it cost 10% less than the largest comparable setup[6] which was built by Professor Terzaghi at the Massachusetts Institute of Technology at Cambridge, in the late 1920's, when building costs were much lower. The present investigation benefited from the experiences of Professor Terzaghi.

The reinforced concrete testing tank (see Fig. 1) is housed in a timber frame enclosure. Its capacity is 70 cu yd. A monorail is suspended from the roof of the stadium and extends from the interior of the enclosure over soil storage bins (see Fig. 2) and a truck ramp. An electrically operated 3-ton hoist with a $\frac{3}{4}$-yd clamshell bucket runs on the monorail and is used for filling the testing tank with sand backfills and for emptying it of all types of backfills. The testing tank was filled with fluid clay and sand-clay backfills (simulating hydraulically placed backfills) by "mudjacks."

Screened silty clay soil material was stored in bins from which it could be transported when needed along the monorail by the crane and the clamshell bucket to a specially constructed platform shown in Fig. 3. From the platform the dry screened soil is shoveled by hand into the mudjack, is mixed in it with water, and then is pumped into a half-yard dump bucket. The consistency of the pumped mixture is made quite liquid, corresponding to approximately two or three blows on the liquid limit device. The crane transports the filled dump buckets into the enclosure where they are emptied into the tank.

Essentially, the tank (see Fig. 4) consists of a massive buttressed reinforced concrete box which encloses (on three sides only) a space 9 ft deep, 13 ft wide, and 18 ft long. The fourth side is closed by the model flexible bulkhead which is made in three entirely separate vertical sections to minimize the effect of friction between the backfill and the side walls of the tank on the center bulkhead. All conclusions are based on measurements on the center bulkhead. All joints between the three bulkheads, the side walls of the tank, the movable bottom plate, and the concrete floor of the tank are sealed by very flexible rubber loops so that no load transfer can occur through any of these joints.

Each of the three bulkheads receives independent support at three elevations, referred to as the A-level, the B-level, and C-level. The *A*-reaction

[6] "Record Earth-Pressure Testing Machine," by K. Terzaghi, *Engineering News-Record*, September 29, 1932, p. 369.

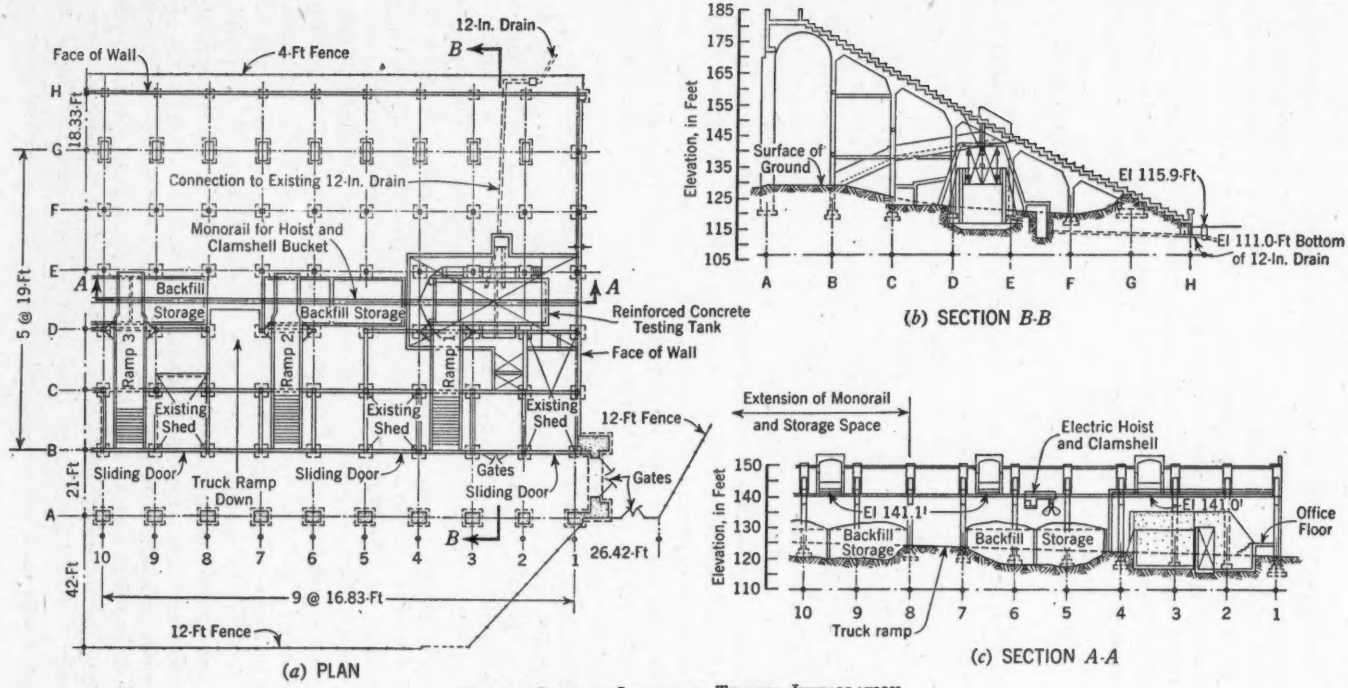

FIG. 1.—GENERAL LAYOUT OF TESTING INSTALLATION

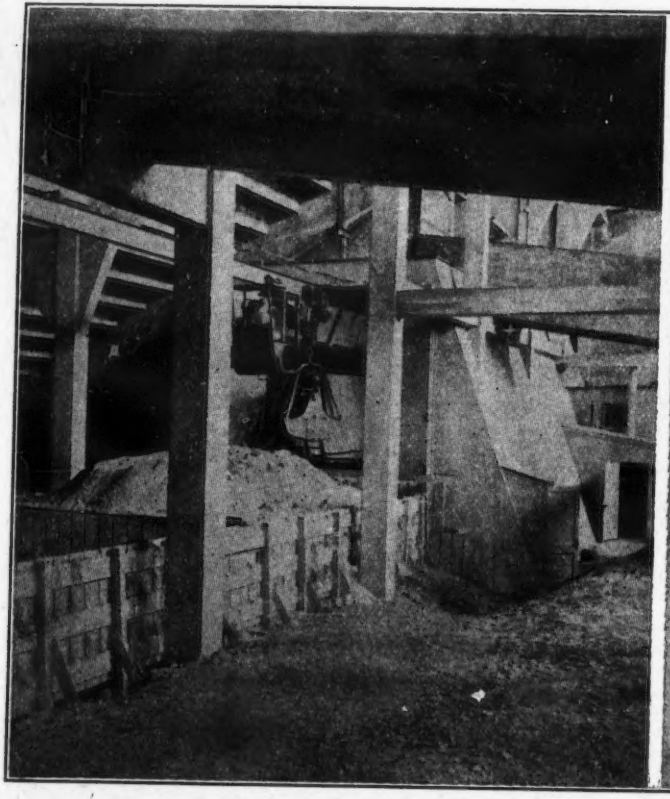

FIG. 2.—MONORAIL, TESTING TANK ENCLOSURE, AND STORAGE BINS

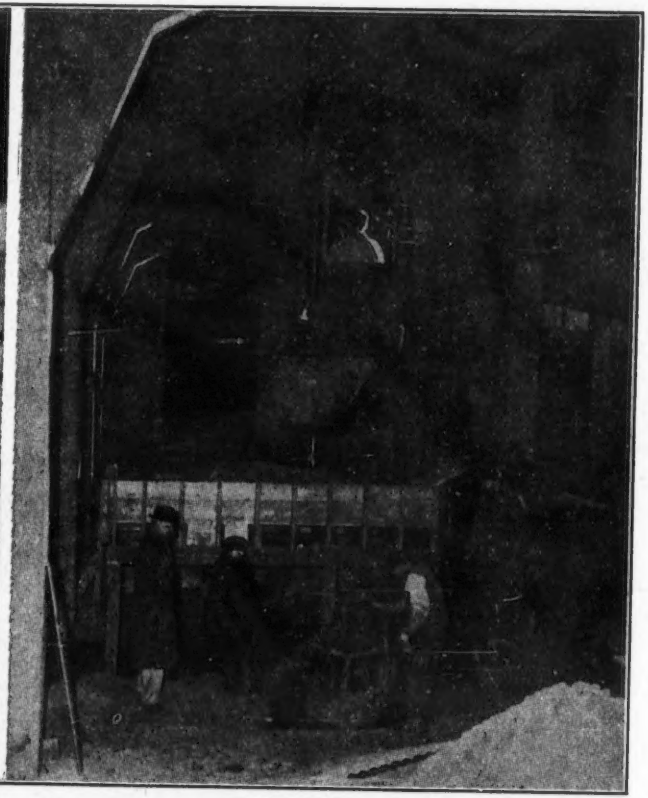

FIG. 3.—OPERATIONS AT THE MUDJACK LOADING PLATFORM

corresponds to the anchor pull and the *B*-reaction and the *C*-reaction are intended to simulate (of course, to a limited extent only) the conditions of support and of varying restraint of the embedded part of a flexible anchored sheet pile bulkhead. To that end, the model bulkheads have been so designed that they cannot undergo any lateral displacement at the C-level but are free to rotate around their vertical support at that level. The *A*-reaction and the *B*-reaction are so designed that controlled lateral displacement of their supports is possible. The bottom plate covering 4 ft of the floor of the tank can be made to yield vertically next to the B-support, when the latter is released—to simulate soil displacements in the field. Such displacements of a backfill in the field are induced when passive resistance of the soil at the toe of the bulkhead is developed after deformation. The free span between the A-level and the B-level is 6 ft, which corresponds to a model scale of approximately 1:5. At the points P (see Fig. 4), six pre-stressing rods are provided (two for each bulkhead) for purposes of calibration and to induce additional deflections.

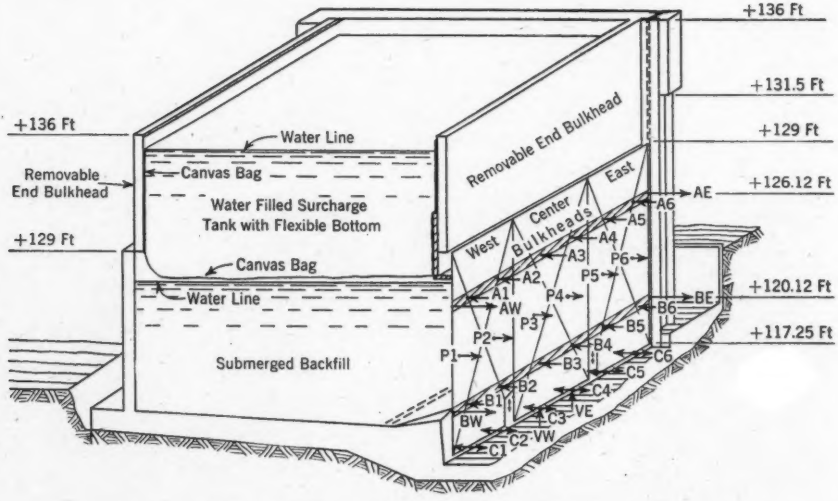

FIG. 4.—SECTION THROUGH THE NORTH END OF THE TESTING ASSEMBLY

The upper part of the testing tank consists of two parallel longitudinal reinforced concrete walls 7 ft high. The space between these walls can be closed at both ends by removable steel bulkheads easily transported along the monorail by the overhead electric crane. A canvas bag fits into the space thus formed, which can be filled with as much as 6 ft of water thus providing a uniformly distributed surcharge over the entire surface of the backfill.

A schematic drawing of the general assembly is presented in Fig. 5. The structural members C1, C2, C3, C4, and M1 form part of the finished testing tank. Places designated DS indicate the center lines of the six supports of the three bulkhead sections at two different elevations. These supports are combined with dynamometers and transmit the fill pressure to the two

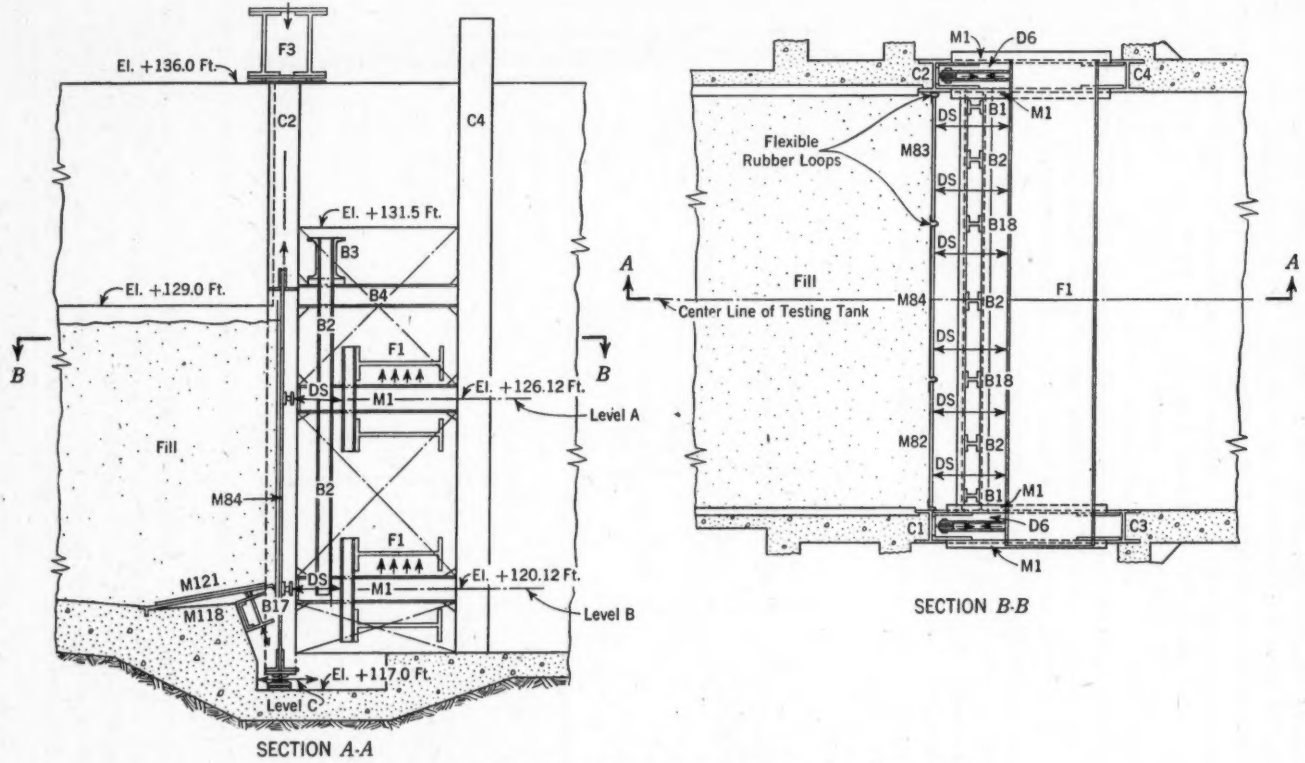

FIG. 5.—GENERAL ASSEMBLY OF STEEL BULKHEAD AND THE SUPPORTING FRAME

main supporting frames marked F1. Designations M82, M83, and M84 indicate the three bulkhead sections; and B17, M118, and M121, the movable base that supports the fill. Designations B1, B2, B3, B4, and B18 constitute the supporting frame for measuring deflections, which was provided with supports independent of those designed to take care of the bulkhead reactions.

The vertical reactions were measured on the central bulkhead with two Carlson pressure cells embedded below the C-level in the concrete floor of the testing tank. Overhead beams were used to suspend the bulkheads only during their erection. Bulkhead deflections and displacements were measured all along the vertical center lines of each bulkhead at elevation intervals of $4\frac{1}{8}$ in. by mechanical dial extensometers reading to 0.001 in.

Electric resistivity strain gages (SR–4) were used in the construction of the specially designed dynamometers[7] which measured the reactions at the bulkhead supports. The same type of gage was used to measure the bending strains across the entire width of the center bulkhead and at every $4\frac{1}{8}$-in. interval of elevation.

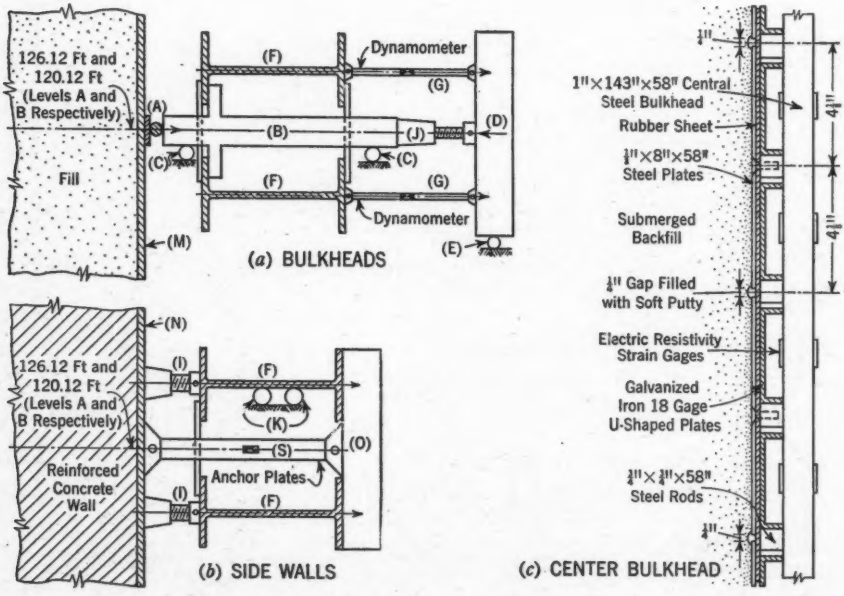

FIG. 6.—VERTICAL SECTIONS THROUGH BULKHEADS AND SIDE WALLS, SHOWING INSTRUMENT SETUP

Figs. 6(*a*) and 6(*b*) are diagrams indicating how apparatus was arranged at levels A and B, for measuring reactions, compensating for temperature variation, pre-stressing members, and controlling the yielding of supports. The main horizontal supporting steel girders (F) are anchored back to the web (N) of the steel column forming an integral part of the reinforced concrete tank. This

[7] "Use of Electric Resistivity Strain Gages Over Long Periods of Time," by Gregory P. Tschebotarioff *Proceedings*, Soc. for Experimental Stress Analysis, Vol. III, 1946, No. 2, p. 47.

web is located in the same plane with the steel bulkheads (M). As a result of this arrangement, the length of all compression members of the supports is equal to the length of all tension members. Since they are all supported on rollers, temperature deformations of these supports will permit them to move without producing any motion of the bulkhead itself. This phenomenon is important since otherwise active and passive earth pressure would be brought into play alternately. The jack (J) shown in Fig. 6(*a*) is used to release the support when so desired; the jacks (I) in Fig. 6(*b*) are used for pre-stressing and balancing purposes only. After the erection was completed, it was found that no frictional or other resistances to lateral motion could be measured in the entire setup.

The special arrangement illustrated in Fig. 6(*c*) permitted the fixation of strain gages on both faces of the central steel bulkhead including the face exposed to the submerged backfill. The result of this design was that the earth and water pressures were transmitted to the main steel bulkhead (1 in. thick) as concentrated line loads $4\frac{1}{8}$ in. apart in a vertical direction. In so far as the bending moments were concerned, for all practical purposes this combination was equivalent to distributed loading, since the span between supports A and B was 72 in. The steel plates ($\frac{1}{8}$ in. thick by 8 in. wide) in direct contact with the backfill were screwed to the main bulkhead along their horizontal center line only and were free to slide at their ends on the Koroseal rubber sheet they covered. It was surmised that this arrangement would have practically no effect on the general rigidity of the bulkhead so that the moment of inertia of the entire system would not be noticeably different from that of a solid steel plate 1 in. thick. This assumption proved to be correct as shown by calibration measurements.

The interior of the tank, during repairs to the insulation of the gages on the center bulkhead, is shown in Fig. 7. Five pairs of strain gages were spaced vertically on $4\frac{1}{2}$-in. centers, on the center bulkhead. The readings of these gages were averaged electrically in two and three pairs, respectively.

The control room is shown in Fig. 8. The south upper removable end bulkhead, the reference system for deflection measurements, and the A-level bulkhead supporting construction are shown in the background.

D. Soils Used for the Tests

The grain-size distribution curves of the sand, the clay, and the sand-clay mixture used as backfill, as determined by the standards of the American Society for Testing Materials (ASTM), are presented in Fig. 9. The liquid limit (w_L), of this silty clay was 30%; the plastic limit (w_P), 23%; and the plasticity index (I_P), 7%. Hans F. Winterkorn, Assoc. M. ASCE, made a further mineralogical and physicochemical study of the red-colored clay in his Soil Physics Laboratory at Princeton University. The results showed that the clay mineral was almost entirely kaolinite. Professor Winterkorn further stated that this New Jersey red clay soil was: "* * * well leached, contains few, if any, exchangeable bases, representing mainly an H-clay, and consequently is microbially quite sterile and chemically quite stable." The clay is a product of shale weathering, and water absorption data indicated a

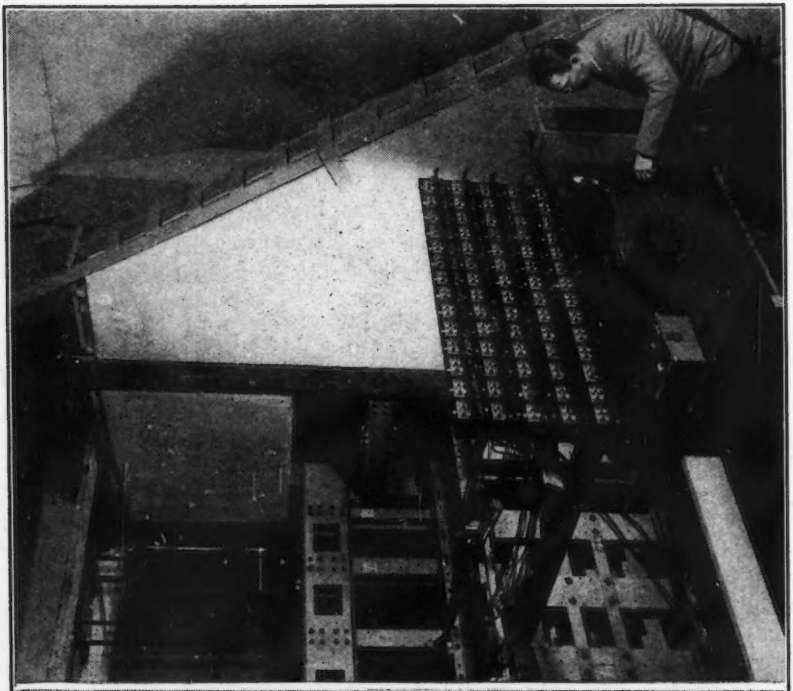

FIG. 8.—CONTROL ROOM

FIG. 7.—STRAIN GAGE INSTALLATION

clay content of about 30%—that is, higher than that obtained by standard ASTM sedimentation tests.

Direct and triaxial shear and unconfined compressive strength tests were performed in the writer's Soil Mechanics Laboratory at Princeton University.

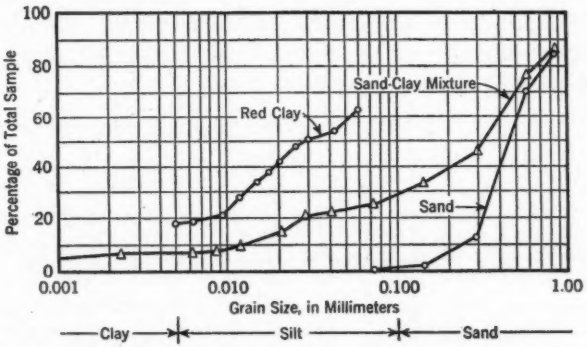

FIG. 9.—SOILS USED FOR BACKFILLING PURPOSES DURING THE MODEL TESTS

Both direct and triaxial shear tests indicated that the sand in a loose condition corresponding to the densities prevailing in the backfill during the tests (from 92 to 96 lb per cu ft dry weight) had an angle of internal friction varying from 32° to 36°. Consolidated rapid shear tests performed on the clay placed in the laboratory testing devices in an originally fluid condition gave an angle of internal friction of approximately 17°. The corresponding value for the sand-clay mixture was approximately 32°. Further shear tests were performed on undisturbed samples from the backfill.

In addition, it was attempted to duplicate in the laboratory the consolidation processes that occurred in the backfill of the testing tank. For this pur-

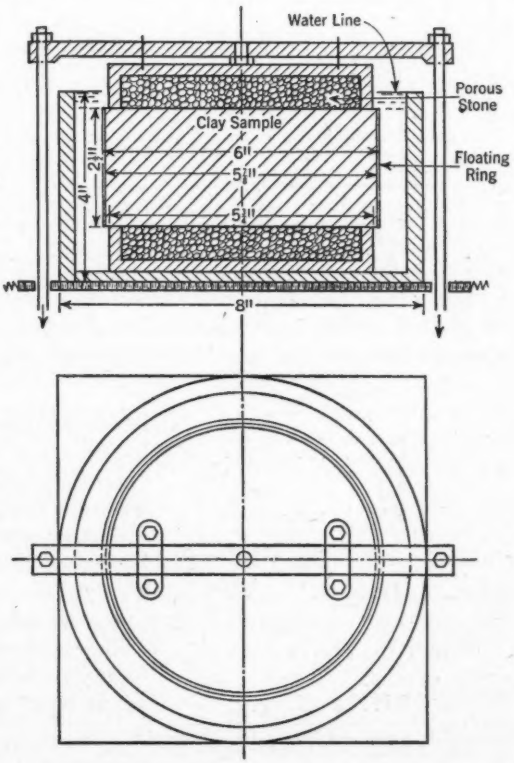

FIG. 10.—OVERSIZE FLOATING RING CONSOLIDOMETER

pose two oversized floating ring-type "consolidometers," 6 in. in diameter, were designed by the writer (see Fig. 10). By use of this device the original fluid backfill material can be consolidated in the laboratory under any desired pressure. The consolidated product can then be cut into 1-in. by 1-in. by 2-in. specimens for the unconfined compressive strength tests. A few tests were also made with still larger floating ring consolidometers (12 in. in diameter) which permitted cutting from the consolidated clay samples 2 in. by 2 in. by 4 in. for unconfined compressive strength tests.

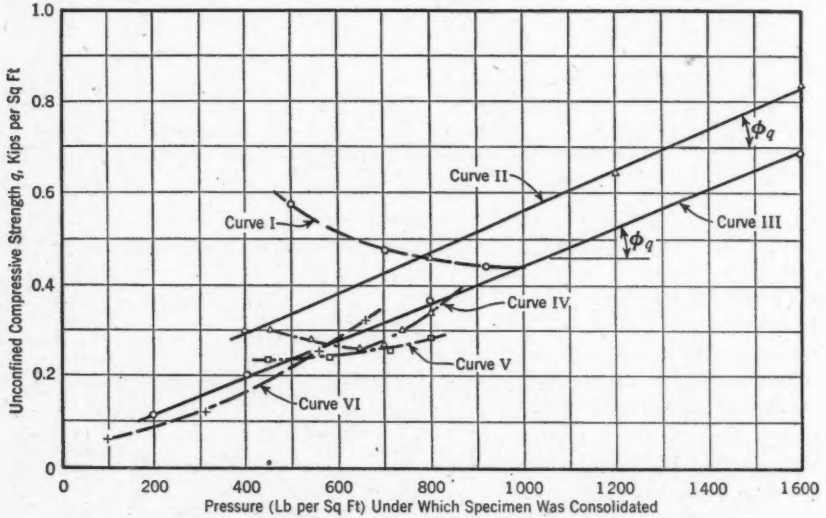

FIG. 11.—TESTS FOR COMPRESSIVE STRENGTH OF CLAY AND SAND-CLAY MIXTURES

The unconfined compressive strengths of both clay and sand-clay mixtures are plotted against the consolidation pressure, in Fig. 11. Curves I and II represent the mixtures, and other descriptions are:

Curve	Description
	Sand-Clay Mixture, Consolidated in—
I.....	Testing tank; sample under its own weight, plus a surcharge of 375 lb per sq ft
II.....	Oversized floating ring consolidometer (laboratory sample)
	Clay Only, Consolidated in—
III.....	Oversized floating ring consolidometer (laboratory sample)
IV and V.....	Testing tank; sample under its own weight, plus a surcharge of 375 lb per sq ft
VI.....	Testing tank; sample under its own weight

The results obtained from tests on laboratory specimens (curves II and III) give straight lines with identical slopes (the angle of which is referred to

as ϕ_q). The undisturbed samples from the backfill, however, do not follow a similar pattern even though their unconfined compressive strengths were plotted in Fig. 11 against consolidation pressures smaller than the actual weight of the overburden, some allowance having been made for effects of friction against the walls of the tank. During the tests with the clay and with the sand-clay mixtures, and after their consolidation was completed under 6 ft of water surcharge, the unconfined compressive strength and the water contents of both types of backfill were found to have an almost constant value throughout the entire depth. In the case of the sand-clay mixture the unconfined compressive strength was even higher in the upper than in the lower layers. The condition observed in the clay backfill may have been caused during a stage of drainage by some evaporation through the 8-in.-thick sand mat overlying the clay. In the test with the sand-clay mixture a similar result may have been obtained because some segregation appears to have occurred during the placing of the fluid backfill so that the upper layers had a higher content of fines and, correspondingly, a somewhat higher unconfined compressive strength at the same water content.

Check tests of long duration (5 months) were performed in consolidometers 15 in. in diameter, to determine whether any of the observed phenomena could be attributed in part to the increased structural strength that results from "aging" of the clay. With this type of clay no effects were observed of the kind described by Professor Terzaghi in 1946[8] in reporting on laboratory developments of his earlier related studies of natural deposits.[9]

E. Investigation Techniques

Two independent calibrations of the testing apparatus were performed. The first calibration was made with the pre-stressing rods P (Fig. 4) which induced bending strains in the bulkhead both before and after the flexible rubber loops and the outer steel plates ($\frac{1}{8}$ in. thick by 8 in. wide) were mounted on the central bulkhead. The results of this calibration showed that the bending-strain readings were identical both before and after these plates were mounted, thus proving that the stiffness of the central bulkhead had not been noticeably increased by the special construction illustrated in Fig. 6(*c*).

The second type of calibration was performed by filling the testing tank with water. Fig. 12 shows the bending strains measured on the central bulkhead during the water calibration. The readings were sufficiently accurate to permit determination of the shears V by the formula:

$$V = \frac{dM}{dx} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which M denotes "bending moment." A smooth curve was passed through the points obtained and this in turn permitted the determination of the pressures p by use of the formula:

$$p = \frac{dV}{dx} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

[8] "Pitfalls of Soil Mechanics," by Karl Terzaghi, *The News*, Philadelphia Section, ASCE, Philadelphia, Pa., May, 1946.

[9] "Undisturbed Clay Samples and Undisturbed Clays," by Karl Terzaghi, *Journal*, Boston Soc. of Civ. Engrs., July, 1941, p. 211.

A fairly reasonable agreement was obtained between the actual and the computed pressure distribution. This method of pressure computation, however, was possible only when strain readings were taken over relatively short periods of time not exceeding a few days, since otherwise too great a scattering

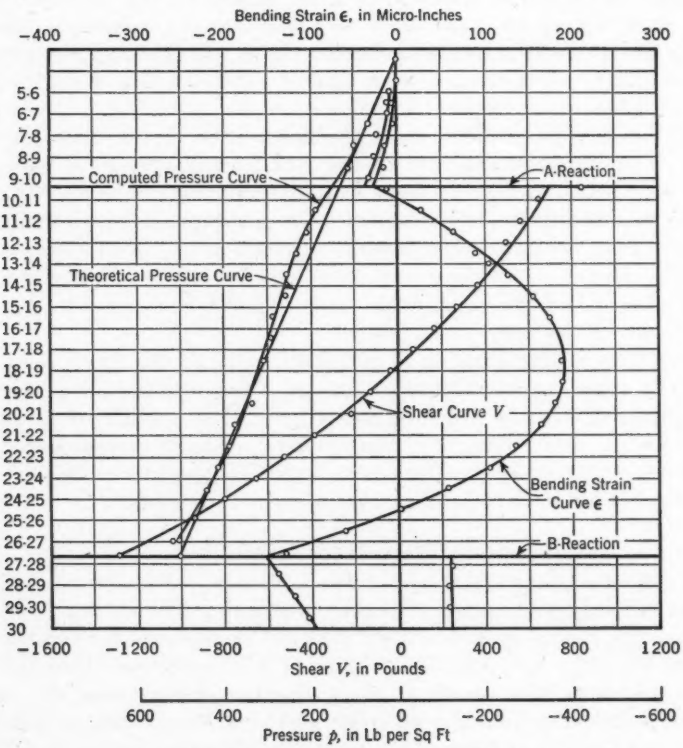

FIG. 12.—CALIBRATION CURVE (8 FT OF WATER)

of the bending-strain readings occurred to permit the double differentiation procedure. Furthermore, reasonably accurate values could be obtained only for the central part of the span between the A-support and the B-support. Within a foot or so from these supports, where the slope of the bending curve changes rapidly, this procedure was not reliable. Nevertheless, in some cases (for instance, for the determination of lateral pressure variation due to increments of surcharge loading), this procedure of lateral pressure computation from observed bending strains of the bulkhead disclosed certain important facts which otherwise could not have been ascertained (see subsequently in Part II, Section D).

The calibration studies permitted the determination of the accuracy of readings taken with various measuring units of the testing assembly. As a result of the calibration it can be stated that the measured reactions on the central bulkhead could be determined with an accuracy of $\pm 10\%$ or better,

the A-level reactions having a slight tendency to read somewhat too high and the B-level reactions having a slight tendency to read too low.

The most reliable type of readings were the bending-strain and the deflection readings. This follows from the nature of this type of measurement, from the calibration tests, and from routine checks of the bending strains and of the deflections against each other. Such checks were performed for all important test stages. Deflections were computed from the measured bending strains by the elastic weight method. The computed deflections compare very favorably to the measured deflections, as a rule not deviating by more than approximately 5%. This is an additional indication of the reliability of the bending-strain readings recorded in this investigation.

The electric resistivity strain gages used in this study permitted measurements of a type which otherwise would have been impossible. Strains of a millionth of an inch per inch (for example, stresses of 30 lb per sq in. in steel) can be detected under favorable conditions. Nevertheless, these gages are extremely delicate and require special techniques of handling. The difficulties encountered and the methods developed to overcome them, have been described elsewhere.[7] As a final result, these gages were adapted to the purposes of the investigation quite satisfactorily. A calibration check was made after each test to permit discarding the readings of gages that had developed signs of instability. It was found advisable to install at least 50% more gages than needed, since approximately one third of them were likely to go out of order during prolonged tests. Tests longer than 6 months appear inadvisable.

The two pressure cells for the measurement of vertical reactions under the center bulkhead performed satisfactorily. However, at low pressures, their readings could be used only as an indication of trends—not of precise values—because the cells had a somewhat higher load capacity than actually proved necessary.

The Carlson pore pressure cells were calibrated and checked for continued stability of readings in a water filled standpipe, 14 ft high. Only cells that performed well were embedded in the backfill. Some trouble developed at first because the cells built up to pressure in the backfill, but did not subsequently show any decrease when the pressure was relieved. This condition was traced to the clogging of the too fine porous stones. After the stones were replaced by a coarser variety the pore pressure cells performed very well and provided valuable data.

Test results were compared mainly by the use of the following four values (see Fig. 13):

(1) The upper reaction A, per foot of width, at the anchor level A;
(2) The lower reaction B, per foot of width, at level B;
(3) The total pressure per foot of width of the bulkhead:

$$R = A + B - C \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

(4) The maximum bending strain ϵ_o of an equivalent beam freely supported at levels A and B.

The significance of the value ϵ_o is illustrated by Fig. 13. The maximum positive bulkhead bending strains that occur approximately midway between

the A-support and the B-support and the maximum negative bending strains at the B-support are very strongly affected by any displacement of these supports. For this reason neither one of these values, taken by itself, can be used for the estimation of pressure intensity and distribution along the span. An accurate control of the displacements of the supports is not practicable; nor is it possible to reproduce "field" conditions by such control since innumerable variations of the conditions of restraint of the lower embedded parts of bulkheads are possible in the field. For the foregoing reasons the writer decided to introduce the value ϵ_o for his analysis. It represents the maximum bending strain of an equivalent beam freely supported at levels A and B. Therefore the value of ϵ_o is not affected by any displacements of the supports and depends only on the magnitude of the total pressure acting against the span A–B of the bulkhead and on the distribution of pressure along that span.

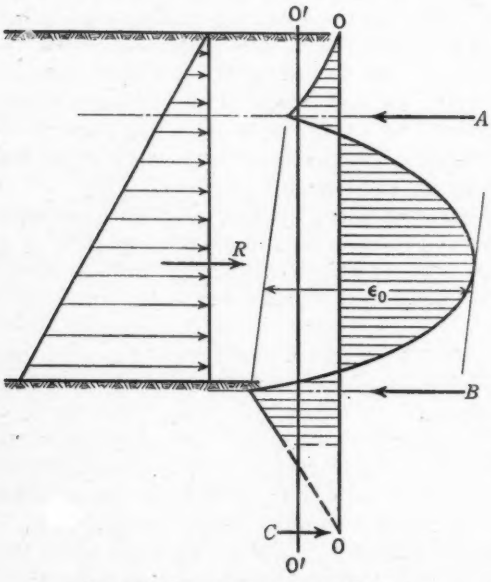

FIG. 13.—SIGNIFICANCE OF THE TERM ϵ_o

In order to compare the test results further the so-called hydrostatic pressure ratios K were computed for each of the preceding four values (A, B, R, and ϵ_o) for all stages of the tests. The corresponding K-values are referred to as the K_A-value, the K_B-value, the K_R-value, and the K_ϵ-value, respectively. As shown in Fig. 13 the K_A-value is equal to the ratio between the actually measured value of the reaction A and the value of A computed for a completely frictionless fluid of unit weight equal to that of the backfill. The K_B-value, the K_R-value, and the K_ϵ-value are computed in a similar manner. All the K-values thus obtained are compared to the theoretical K-values computed on the strength of different types of laboratory tests and of different formulas and assumptions concerning the shearing strength which is effective in reducing lateral pressures.

Possible arching effects were estimated by comparing the measured K_ϵ-values to the conventional K_ϵ-values. Additional information on this point was obtained by comparing the measured K_ϵ-values to the measured K_R-values since arching produces only a redistribution of pressure which therefore should affect the bending moments only and not the total pressure. The actual measured values of the negative bending strains at the A-support were also considered in this connection.

The K_A-coefficient and the K_B-coefficients were further used for comparison of lateral pressures exerted by the backfill in its upper (conventionally assumed as tensile) zone and in its lower (compressed) zone limited by a rigid boundary below it.

PART II. THE PRINCIPAL FINDINGS OF FIRST TEST SERIES

A. Structural Effect of Sand Dike

Tests were performed with a sand dike provided with a berm 1 ft wide along line 6–6, Fig. 14, with a sand blanket of a thickness equal to the height of the bulkhead (line 8–8, Fig. 14), and with a sand blanket only one half as thick (line 9–9, Fig. 14). The lateral pressure of the sand dike alone produced bending strains ϵ_2 equal to approximately 60% of the values corresponding to a backfill composed entirely of sand.

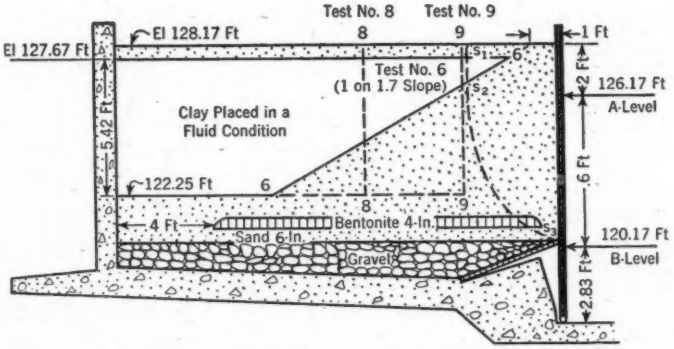

Fig. 14.—Structural Effect of a Sand Dike

The use of a sand dike sloping away from the bulkhead on a natural (1:1.7) slope was found to be fully effective in reducing the fluid lateral pressures transmitted to the bulkhead from the unconsolidated fluid clay backfill behind the dike. The pressures against the bulkhead were no greater than those exerted by a backfill composed entirely of sand.

The interposition between the fluid clay backfill and the bulkhead of a vertical sand blanket with a width equal to the bulkhead height was found to be just as effective as the interposition of a sand dike. When the width of the blanket was equal to one half of the bulkhead height, it was only approximately one half as effective, and it was completely ineffective when its width was equal to one tenth of the bulkhead height. In that latter case the lateral pressures transmitted to the bulkhead from the unconsolidated clay backfill were no smaller than those of a fluid.

The foregoing statements should be taken only as indications of order of dimension, accurate to approximately $\pm 10\%$.

Consolidation of the clay did not further decrease lateral pressures in the case of the sand dike. Fig. 14 illustrates that only a small and the least ef-

fective part of the likely logarithmic spiral slip surface (S_1–S_2, Fig. 14) lies within the clay. The same observation concerning lateral pressures was made in the case of the wide sand blanket (8–8, Fig. 14). In that case the entire probable slip plane lies within the sand mass; this could not happen when the

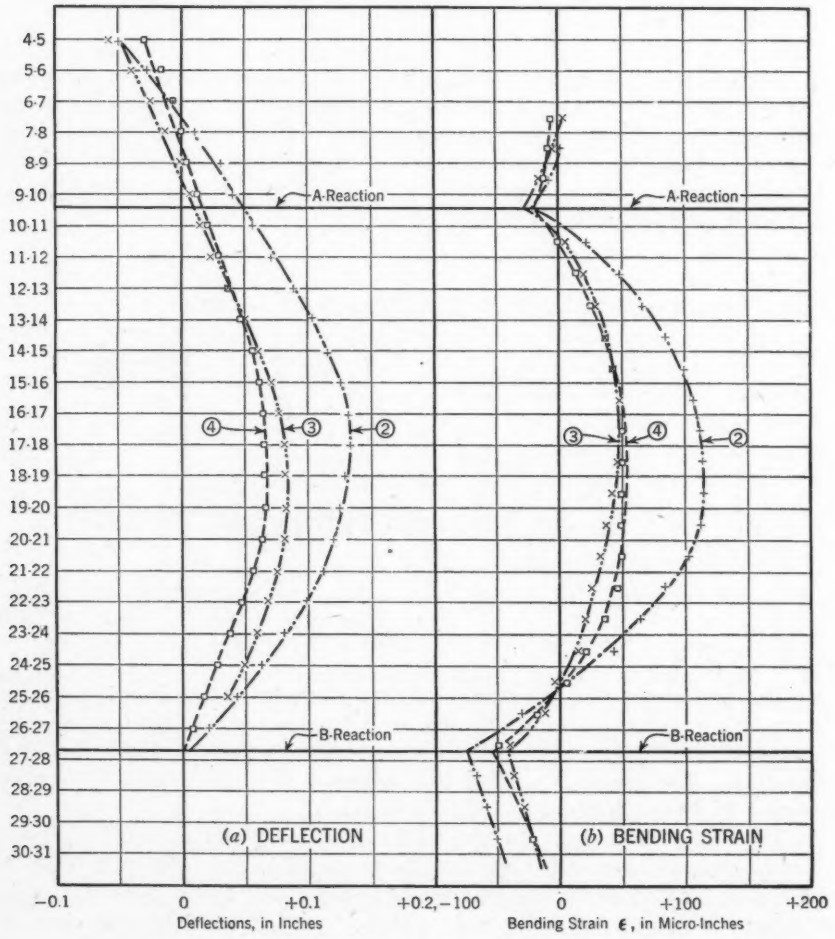

FIG. 15.—COMPOSITE CURVES FOR THE COMPLETELY FLUID CLAY BACKFILL CONDITION, MINUS 8 FT OF WATER

thinner sand blanket (9–9, Fig. 14) was used, and therefore its action was less effective at first. Subsequent consolidation of the clay backfill correspondingly decreased the lateral pressures. The foregoing analysis is somewhat different from the one advanced by the author of the fourth Symposium paper.

It should be noted that a bentonite layer 4 in. thick had been placed over the sand and gravel drainage layer during a preceding test. Although originally placed at a water content well above its liquid limit, the layer was rapidly consolidated so that it had an unconfined compressive strength of approximately 200 lb per sq ft. As a result, its presence does not seem to have affected the action of the sand dike.

During a preceding test with a backfill entirely composed of sand, the presence of a bentonite layer (placed as shown in Fig. 14) had the effect of reducing the lateral pressures somewhat, in comparison with those in another test in which no bentonite layer was present. This observation may be explained by the lateral expansion of the sand backfill as a result of the consolidation of the bentonite, which expansion may have reduced the lateral pressures.

Curve 2, Fig. 15(*b*), shows the component of the bending strains produced by the buoyed weight of the solid fraction of the fluid clay backfill during test No. 3 when the entire backfill was clay; curve 3 shows the same values for test No. 5 when the backfill was composed entirely of sand; and curve 4 gives almost identical values obtained during test No. 6—when the sand dike was backfilled with fluid clay. The effectiveness of the sand-dike action is clearly shown.

This finding is confirmed by a similar relationship of the deflection curves of the center bulkhead during the three tests just mentioned (see Fig. 15(*a*)).

B. Lateral Pressures of Consolidating Clay Backfills

The use of a sand dike can effectively reduce the lateral pressures of a fluid clay backfill, but it cannot speed up the consolidation of the backfill itself and thereby increase its power to support vertical loads.

(*1*) *Accelerated Consolidation of Fluid Clay Backfills by Use of Prefabricated Horizontal Sand Drains.*—The consolidation of originally fluid clay backfills can be accelerated effectively by the use of a system of prefabricated horizontal sand drains (for example, burlap cylinders filled with sand) connected to well points. This method was successful during one of the tests.

Vertical sand drains have been used successfully for some time; but,[10] in so far as lateral pressures of backfills are concerned, they have some disadvantages. First, they cannot be installed until most of the backfilling is completed. Second, each drain must be pumped out or bailed out individually if uplift pressures are to be neutralized temporarily within the entire mass of the backfill in order to increase its effective weight, and thus to accelerate consolidation still further.

An interconnected system of horizontal prefabricated sand drains does not have the aforementioned disadvantages and has some further merits as compared to horizontal sand blankets—the continuity of which is likely to be destroyed by mud wave formation during backfilling operations. The installation of such a system of prefabricated drains in the field, however, is

[10] "Studies of Fill Construction Over Mud Flats Including a Description of Experimental Construction Using Vertical Sand Drains to Hasten Stabilization," by O. J. Porter, *Proceedings*, Highway Research Board, National Research Council, Washington, D. C., 1938, Pt. II, pp. 129–141.

likely to present some difficulties and to be expensive. Fig. 16 shows the writer's conception of a possible construction procedure.

(*2*) *"At Rest" or "Consolidated Equilibrium" Lateral Pressures.*—During the consolidation (in a submerged state) of an originally fluid clay backfill, investigators observed that, in the backfill, there was a decrease of pressures,

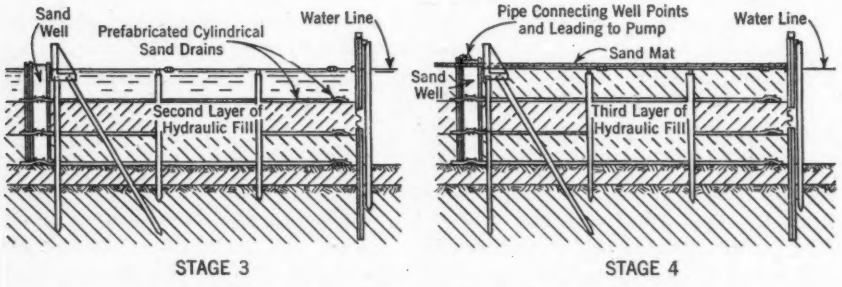

FIG. 16.—INTERCONNECTED LAYER SYSTEM OF PREFABRICATED HORIZONTAL DRAINS

and that, in the bulkhead itself, there was a simultaneous decrease in bending strains, deflections, and horizontal reaction pressures. These decreases continued until the consolidation was completed and an "at rest" or (to use a possibly more appropriate term) a "consolidated equilibrium" condition was reached.

The decrease of bending strains during consolidation of the clay backfill is illustrated by Fig. 17(*a*). The progress of the test is recorded by stages, as follows:

Stage	Date (1944 and 1945)	Description
1	December 13	Initial position of bulkhead
3	December 21	Placing backfill plus sand mat completed to El. 128.17; flooded
4	December 27	Last reading before drainage
8	January 29	First reading of the first 6-ft water surcharge
10	February 20	Last reading of the first 6-ft water surcharge

Test stages 3 to 4, Fig. 17, correspond to backfill consolidation under its own weight and test stages 8 to 10, Fig. 17, correspond to the consolidation of the backfill under a surcharge of 375 lb per sq ft. Fig. 17(*b*) shows a similar decrease of deflections during the same test stages.

Conventional stress-strain equations have no meaning in this case, since the increment of strain has a negative value, whereas both the increment of stress that is effective in reducing lateral pressures and the modulus of elasticity have positive values.

According to one current popular concept,[11,12] outward motions of more than 5% of the height of the bulkhead are needed to mobilize fully the shearing

[11] "A Fundamental Fallacy in Earth Pressure Computations," by Karl Terzaghi, *Journal*, Boston Soc. of Civ. Engrs., April, 1936, p. 87.
[12] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, p. 96.

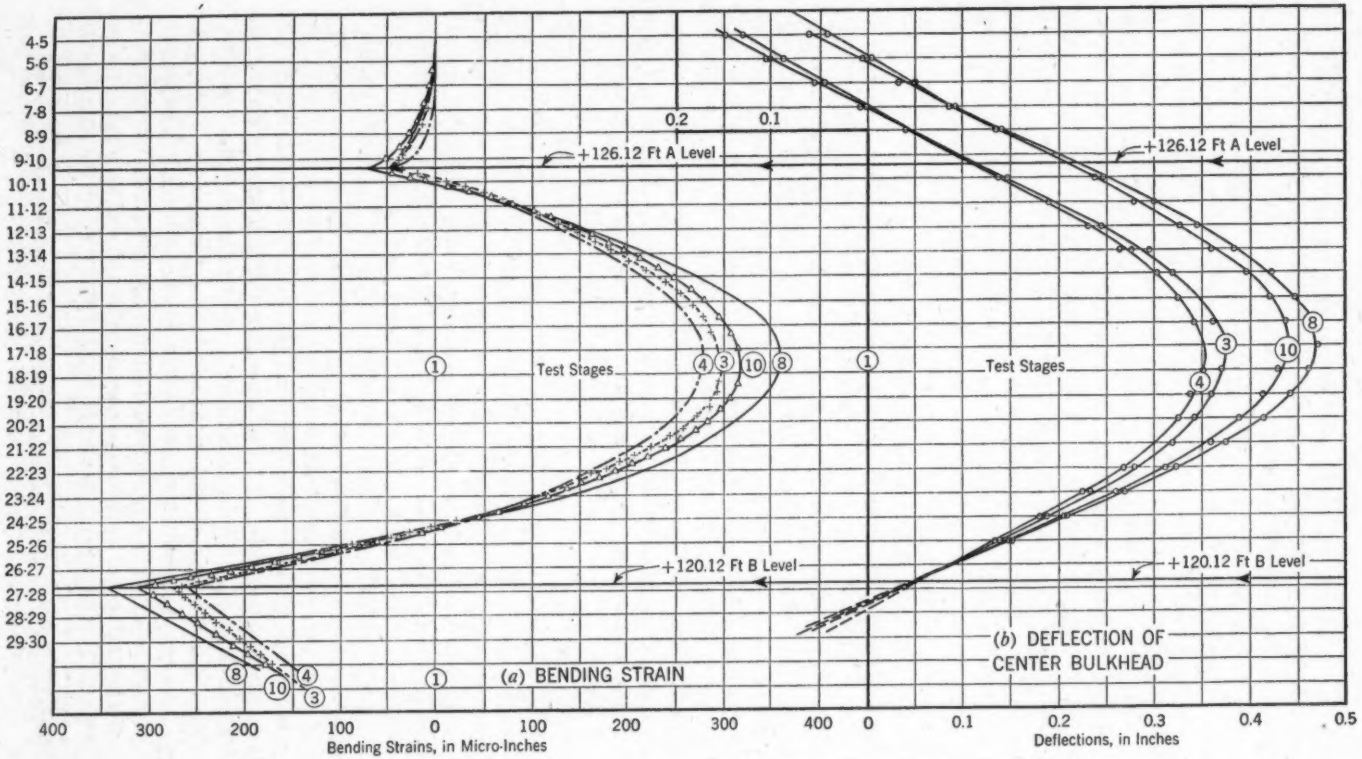

FIG. 17.—DECREASE IN BENDING STRAINS AND DEFLECTION DURING CONSOLIDATION (TEST NO. 3)

strength of a clay soil which is effective in reducing lateral pressures. The Princeton tests show that this concept applies neither to plastic backfills behind flexible bulkheads nor to their "consolidated equilibrium" pressures. The latter are smaller than fluid pressures; and no induced outward motion of the bulkhead is needed to achieve the corresponding reduction.

Conventional laboratory methods of determining shearing strength coefficients of soils are based on the strength of soil specimens at failure. There is no reason to assume that the values of the "shearing strength at failure," thus determined in the laboratory, are necessarily identical to the values of the soil shearing strength which are effective in reducing fluid lateral pressures of unconsolidated clay backfills against flexible bulkheads to a "consolidated equilibrium" condition. This pressure reduction occurs without any outward motion of the bulkhead and therefore without the development in the backfill of deformations of the type necessary to produce failure in laboratory specimens. Obviously, this condition does not involve complete utilization of all the latent shearing resistance of the consolidated clay.

During the Princeton tests the over-all K_R-values (see Part I, Section E)—for the "consolidated equilibrium" condition of submerged clay backfills—were found to lie between the limits of 0.70 and 0.80. These coefficients are the K_{s+w}-values defined subsequently in Part II, Section E. They include the weights of both solids and water. The K-coefficients are usually intended to represent the so-called hydrostatic pressure ratio, or the ratio of actual lateral pressure to the fluid lateral pressure of a liquid of a unit weight equal to that of the soil. For naturally consolidated clays the K-values "at rest" have not been known; but they have been estimated by Professor Terzaghi, apparently on the strength of intuitive reasoning, as probably ranging between 0.66 and 0.88.[13]

W. S. Housel,[14] M. ASCE, also has reported measurements on tunnel linings, from which the final value of the ratio of measured lateral pressure to the computed weight of the overburden at the same elevation can be computed. It is approximately 0.66. The driving of the tunnel must have produced appreciable disturbance of the clay in its vicinity; hence this value, similar to the Princeton data, corresponds to a "consolidated equilibrium" condition and not to an "at rest" state of a natural deposit.

(*3*) *Lateral Pressures in Upper Unrestrained Zone of Backfill.*—During the tests an induced outward tilting motion, as much as 1.5% of the height of the bulkhead, did not continue to decrease the "consolidated equilibrium" pressures in the upper zones of either a clay backfill or a sand-clay mixture. This condition was found to be true even when the backfill was overconsolidated by the application of a surcharge and appreciably increased its cohesion (as much as four times) as evidenced in the laboratory by unconfined compressive strength tests on undisturbed samples taken from the backfill. (The unconfined compressive strength was increased from approximately 70 lb per sq ft to approximately 300 lb per sq ft for the clay backfill and from an unknown value to 570 lb per sq ft for the sand-clay mixture backfill.) Since the shearing stress

[13] "Liner-Plate Tunnels on the Chicago (Ill.) Subway," by Karl Terzaghi, in "Earth Pressure and Shearing Resistance of Plastic Clay: A Symposium," *Transactions*, ASCE, Vol. 108, 1943, p. 989.

[14] "Earth Pressure on Tunnels," by W. S. Housel, *ibid.*, p. 1045, Fig. 41.

and shearing strain of plastic clays have a parabolic relationship, and since no evidence was found of an increasingly effective shearing strength at 1.5% outward motion of the bulkhead, the writer considers that no further appreciable utilization of strength could be encountered in the upper backfill zone at larger induced displacements—say, at an outward motion of 5% of the height of the bulkhead. In addition, such large motions could not be normally induced in the field and therefore were not included in the Princeton tests.

Since these findings and conclusions were presented at the January, 1947, Annual Meeting of the Society, the writer has learned of confined triaxial tests performed in 1934 in Holland (from a letter dated June 24, 1947, from E. C. W. A. Geuze, chief engineer, Research Department, State Laboratory of Soil Mechanics, Delft, Holland). A decrease of lateral pressure was observed then during the consolidation of undisturbed clays which was accompanied by a decrease of the diameter of the specimen. The tentative conclusion was also arrived at there that the active pressure of clays should not differ greatly from their "at rest" pressure. These findings were presented in a report to the 1936 "Congrès de l'Urbanisme et du Sous-Sol" in Paris. The report has not yet been published.

Since definite values of lateral pressures were actually measured against the upper part of the bulkhead, this finding shows the fallacy (or, at best, the limitation) of a popular concept illustrated by Fig. 18(*a*), according to which there will be no lateral pressure of a plastic clay backfill to a so-called "critical

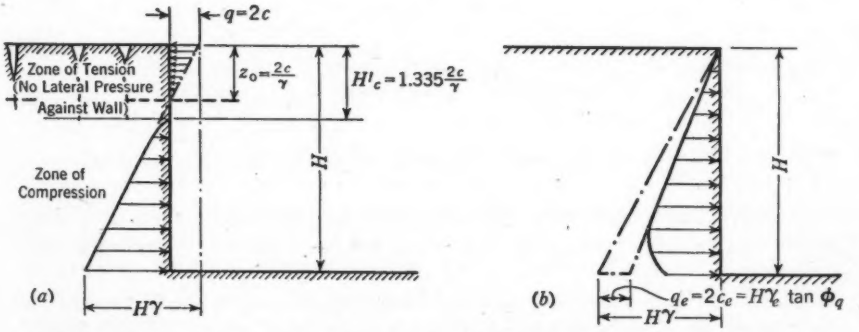

FIG. 18.—COMPARISON OF CONCEPTS CONCERNING LATERAL BACKFILL PRESSURES OF COHESIVE SOILS

depth H'_c"—that is, to the depth at which a vertical cut will stand unsupported.[15] Since plastic deformations and outward bulging always occur in such unsupported cuts, any anchors or struts that prevent bulging should naturally receive some pressure even if the "critical depth" of excavation, or the "critical height" of backfilling, is not exceeded. A quantitatively exaggerated example of this bulging has been presented by Dale T. Harroun,[16] Assoc. M. ASCE. If the critical depth is exceeded, lateral pressures in the upper zone can no longer

[15] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, p. 97.

[16] "Stability of Vertical Embankments," by Dale T. Harroun, *Proceedings*, Highway Research Board, National Research Council, Washington, D. C., 1940, p. 754, Fig. 6.

be governed by the preceding conditions and will depend on the deformation of the entire plastic mass, including that which occurs below the "critical height." After any outward induced bulkhead motion, the unrestrained upper zone of the plastic clay will readjust itself to conditions corresponding to the original consolidated equilibrium and will restore the corresponding pressure.

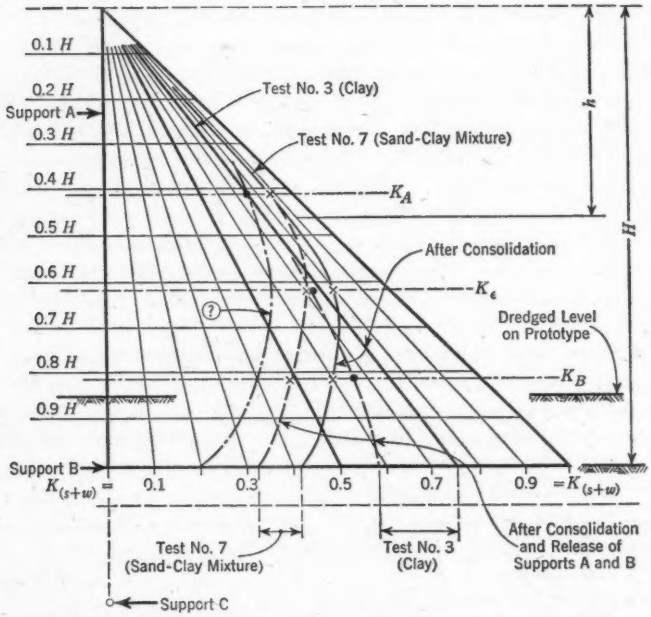

FIG. 19.—ESTIMATED PRESSURE DISTRIBUTION OF CLAY AND SAND-CLAY BACKFILLS

The concept illustrated by Fig. 18(*a*) is apparently intended for use with stiffly plastic clays above water level; but it has not been definitely limited to such conditions.

The writer is not aware of any one having measured, in plastic soils of any consistency, an absence of lateral pressures against the upper "critical height" part of retaining structures with a height greater than the "critical"—that is, of the kind illustrated by Fig. 18(*a*).

The general trend of pressure distribution established during the Princeton tests is illustrated by Fig. 18(*b*). It applies to plastic clay backfills behind flexible bulkheads; and it is valid for material that has been fully consolidated under its own weight as well as for material that has been overconsolidated along the entire length of a bulkhead. The significance of the term q_c will be discussed subsequently in Section B(5).

(*4*) *Lateral Pressures in Lower Restrained Zone of Backfill.*—Induced outward motion of the bulkhead did decrease the "consolidated equilibrium" pressures in the lower zone of the backfill which is restrained by the presence of an underlying rigid boundary. This was particularly pronounced in the

case of a more "rigid" backfill consisting of a sand-clay mixture, as illustrated by Fig. 19, which gives the approximate distribution, with depth, of the K-values determined from the measured K_A-value, K_B-value, and the K_c-value. Since

$$K_h = \frac{p_h}{\gamma h} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

it also gives an idea of the distribution with the depth h of the lateral pressures p_h of a soil of unit weight γ. The dash-dot curve in Fig. 19, marked "?," represents an hypothetical pressure distribution which may occur in the field if the relative dredged level in the prototype is higher than that in the model.

(*5*) *Measured Lateral Pressures of Plastic Cohesive, and of Semicohesive, Backfills Compared to the Results of Laboratory Tests.*—No generally valid relationship could be established between the shearing strength of clays and of sand-clay mixtures at failure (as determined in the laboratory from unconfined compressive strength, and direct shear and triaxial shear tests) and the measured lateral pressures. The conventional formula[17] for total lateral pressure, R, based on the Rankine-Résal theories is

$$R = \frac{\gamma H^2}{2} \tan^2 \left(45° - \frac{\phi}{2} \right) - 2\, c\, H \tan \left(45° - \frac{\phi}{2} \right) \ldots\ldots (5a)$$

and the corresponding formula for unit lateral pressures p_h is

$$p_h = \gamma h \tan^2 \left(45° - \frac{\phi}{2} \right) - 2\, c \tan \left(45° - \frac{\phi}{2} \right) \ldots\ldots (5b)$$

Eqs. 5 refer to fully consolidated soils and state that fluid lateral pressures are decreased as a result of internal friction (expressed by its angle ϕ) as well as by the cohesion c.

No generally accepted procedure has been devised for determining the values ϕ and c for practical use in Eqs. 5. For the same soil the values of ϕ and c may vary greatly depending on the procedure used to determine them in the laboratory.

Studies by Professor Terzaghi[18] and Ralph B. Peck,[17] Assoc. M. ASCE, have resulted in the wide acceptance of a trend to consider that friction (as expressed by laboratory values of ϕ) is inactive in clays, and to take into consideration only the cohesion c, which is assumed equal to one half of the unconfined compressive strength q:

$$c = \frac{q}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Other procedures, nevertheless, are still used occasionally. The writer strove to approach the matter with a completely open mind—to compare results obtained on the strength of different laboratory testing procedures with actual measurements of lateral pressures and to let the facts speak for themselves. These facts were as follows:

[17] "Earth-Pressure Measurements in Open Cuts, Chicago (Ill.) Subway," by Ralph B. Peck, in "Earth Pressure and Shearing Resistance of Plastic Clay: A Symposium," *Transactions*, ASCE, Vol. 108, 1943, pp 1008–1036.

[18] "Liner-Plate Tunnels on the Chicago (Ill.) Subway," by Karl Terzaghi, *ibid.*, pp. 970–1007

a. Consolidated rapid shear tests on undisturbed samples extracted by means of Shelby tube samplers, 3 in. in diameter, from lined bore holes, 4 in. in diameter, in the backfill gave much too high values of ϕ and c for use in Eqs. 5. Lateral pressures computed with these values were too low even if either ϕ or c were neglected.

b. Results of rapid shear tests on originally fluid samples of clay consolidated in the laboratory, when plotted in the conventional manner, gave values of $c = 0$ and of $\phi = 17°$ which, when used in Eqs. 5, agreed approximately with measurements on the model. In the case of the sand-clay mixture the values of ϕ (32°) thus obtained were much too high (approximately twice as high).

c. Unconfined compressive strength tests on the clay samples gave values of c determined from Eq. 6 which, when used in Eqs. 5, were in reasonable agreement with observed pressures as long as the clay was not overconsolidated. Otherwise the c-values were too high. In the case of sand-clay mixtures the c-values thus obtained were too high under all circumstances.

The following expression, developed by the writer, happens to correlate laboratory test results and the measured values of lateral pressure both of consolidated clays and of consolidated sand-clay mixtures (see Fig. 18(*b*)):

$$p_h = \gamma_e h \left(1 - \frac{q_c}{\gamma h} \right) = \gamma h \left(1 - \frac{\gamma_e}{\gamma} \tan \phi_q \right) \ldots\ldots\ldots\ldots (7)$$

in which γ_e represents the effective unit weight of the soil transmitted from grain to grain. In the case of a submerged soil it represents its buoyed dry unit weight.

The significance of the angle ϕ_q can be seen from Fig. 11. Its value is almost identical both for the clay and for the sand-clay mixture. A further discussion of this point appears premature.

It should be emphasized (see Fig. 19) that the consolidated clay and the consolidated sand-clay mixture exerted very similar lateral pressures, although these two types of soil had completely different characteristics when tested to failure by different conventional laboratory procedures (see Fig. 11 and Part I, Section D). The greatest discrepancies between observations and results of laboratory strength tests have been recorded for the sand-clay mixture. This type of material is frequently used for hydraulic backfilling operations.

(*6*) *Possible Use of the Theories of Elasticity and of Plasticity*.—The preceding findings make it appear advisable to examine possible methods of determining lateral pressures which do not relate them to their strength at failure, since under normal field conditions plastic clay and sand-clay backfills cannot be deformed to the point of failure. In the past, suggestions have been made by Lazarus White, E. A. Prentis, and the late George Paaswell,[19] Members, ASCE, that the theory of elasticity be used for the determination of lateral pressures of all soils. However, the knowledge of the value of the Poisson ratio of soils, at least, is necessary for the purpose. There is no known labora-

[19] "Cofferdams," by Lazarus White and Edmund Astley Prentis, Columbia Univ. Press, New York, N. Y., 1940, pp. 75–78.

tory method of determining this value; and usually it is estimated in reverse, from actual measurements.

For the time being, therefore, a more direct experimentally justified approach to design problems appears advisable, as recommended subsequently under Section E. Nevertheless, the adaptation of the theories of elasticity and plasticity to soil problems, so as to permit the development of practically usable methods for the determination of lateral pressure distribution of different types of soils under varying conditions of boundary restraint, should not be considered permanently impossible. It poses a challenge both to theoreticians and to experimenters. New attempts are continuously being made. For instance, W. H. Weiskopf,[20] M. ASCE, has offered a modification of one of the conventional assumptions. He starts with a varying ratio between the modulus of shear E_G and of the modulus of elasticity E, instead of the conventional constant ratio.

C. Phenomena of Arching in Sands

(*1*) *Findings of the Princeton Model Tests.*—With reference to normally placed backfills, no evidence was found of any pressure relief ascribable to vertical arching between the anchor level and the level of the soil below the dredged line. No decrease of bulkhead bending moments was observed corresponding to such arching action, as is sometimes assumed. This finding refers to normally placed sand backfill—material dumped through water so that it slopes away from the bulkhead at a uniform height across the entire width of the tank, at all stages of the backfilling.

Indications were that there was arching under such conditions when the sand is in place at both end supports of a yielding structural unit before its final deflection. For instance, arching in a horizontal direction occurred when the sand was not uniformly dumped across the entire width of the tank. Relatively greater pressures were developed on the bulkhead section against which the sand was placed first, which then could serve as an "abutment" for the horizontal arch in the sand which relieved pressures against the remaining bulkhead sections that were backfilled last.

Arching in a vertical direction did develop after completed backfilling and an additional induced deflection between the supports. This corresponded to the small-scale tests reported by J. T. R. N. Stroyer.[21]

Such arching (in a vertical direction) as did occur was found to be unstable, so that it should not be relied on for design purposes where sand backfills are concerned. A slight release of the bottom plate of the tank near the B-support immediately and strongly increased lateral pressures at all levels.

Indications were obtained, however, of possibly smaller lateral pressures of sand than are usually assumed below the natural ground level against bulkheads.

(*2*) *Other Conceptions and Their Analysis.*—In the case of clean sand it has been held that[22] the actual bending moment in a sheet pile bulkhead, as pro-

[20] "Stresses in Soils Under a Foundation," by Walter H. Weiskopf, *Journal of the Franklin Institute*, June, 1945, pp. 445–465.

[21] "Earth-Pressure on Flexible Walls," by Jens Peter Rudolf Nielsen Stroyer, *Journal*, Inst. C. E., London, Vol. 1, 1935–1936, pp. 94–139 and 550–557.

[22] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley & Sons, Inc., New York, N. Y., 1943, pp. 217–219.

duced by a pressure distribution corresponding to line $as_1t_1b_1$, Fig. 20, will not exceed about one half of the maximum bending moment computed on the basis of hydrostatic pressure distribution (line atb_1, Fig. 20). The writer, among others, shared this view. The total pressure represented by both lines is approximately the same. Therefore, a decrease of positive bending moments below the anchor level can be produced only by a redistribution of pressures over the span beneath the anchor level; or by an increase of pressure above the anchor level with a resulting increase of negative bending moments at that level.

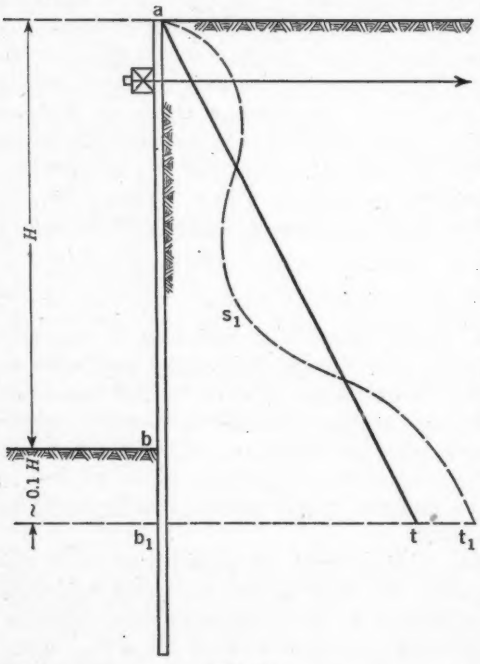

FIG. 20

Fig. 21 illustrates the changes that occur in the maximum bending moments of a freely supported beam, depending on six possible limit cases of load distribution, the total load remaining the same. It shows that a redistribution of pressures over a span of the type given by Fig. 20 cannot alone produce a 50% reduction in the bending moments; at best it can produce a reduction of from 20% to 25%. The Princeton tests show that passive pressures above the anchor level could not have developed a 50% total reduction. Such pressures are not actually developed, presumably because of the progressive yielding of the anchor support, which occurs simultaneously with the deflection of the bulkhead itself (see Figs. 15(*a*) and 17(*b*)). The support did yield in spite of the fact that the A-support was somewhat more unyielding than a real anchor would have been.

Current opinions concerning a strong effect of arching on the decrease of bending moments in the sheet piling have been strongly influenced by the writings of R. Stroyer,[21,23,24] in England. In suggesting the use of his graphs for actual design, the fact has been often obscured[25] that some of his recommendations were purely speculative and that others were based on small-scale model tests which did not reproduce actual backfilling conditions.

In their turn, Mr. Stroyer's conclusions have been influenced by recommendations of the Danish Society of Engineers, first issued in 1926[23] and then

[23] "Earth-Pressure on Flexible Walls," by Rudolf Nielsen Stroyer, *Minutes of Proceedings*, Inst. C. E London, Vol. 226, 1929, pp. 116–134.

[24] "Concrete Structures in Marine Work," by R. Stroyer, London.

[25] "Sheet Piling, Cofferdams, and Caissons," by Donovan H. Lee, Concrete Publications, Ltd., London, 1945, p. 35.

amplified in 1937.[26] As far as the writer could ascertain, these recommendations represented an empirical attempt to take account of the fact that some bulkheads had stood up, whereas according to conventional ideas they should have collapsed.

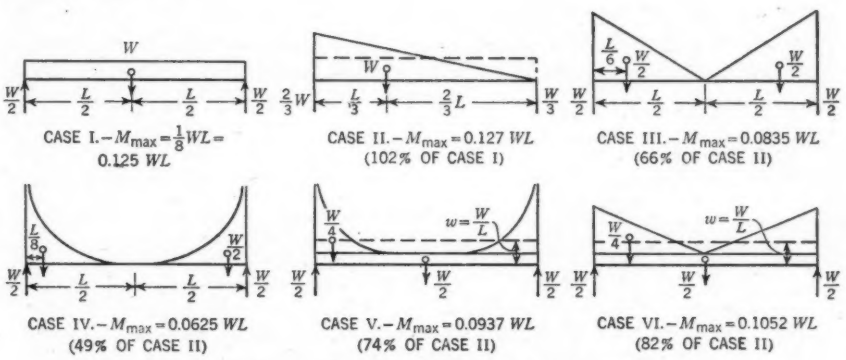
CASE I. – $M_{max} = \frac{1}{8}WL = 0.125\ WL$
CASE II. – $M_{max} = 0.127\ WL$ (102% OF CASE I)
CASE III. – $M_{max} = 0.0835\ WL$ (66% OF CASE II)
CASE IV. – $M_{max} = 0.0625\ WL$ (49% OF CASE II)
CASE V. – $M_{max} = 0.0937\ WL$ (74% OF CASE II)
CASE VI. – $M_{max} = 0.1052\ WL$ (82% OF CASE II)

FIG. 21.—POSSIBLE CHANGES IN MAXIMUM BENDING MOMENTS, THE TOTAL LOAD REMAINING UNCHANGED

The distribution in Fig. 22 produces approximately a 40% reduction of bending moments as compared to conventional diagrams. However, only an approximately 18% reduction is caused by a redistribution of pressures that accompany arching and the remaining approximately 22% reduction results from an assumed decrease of the total pressure. The anchor pulls are not decreased by the Danish distribution, almost the entire pressure reduction occurring along the lower half of the span below the anchor level. This is in general agreement with the Princeton findings, although the shape of the pressure distribution curve in Fig. 22 does not agree.

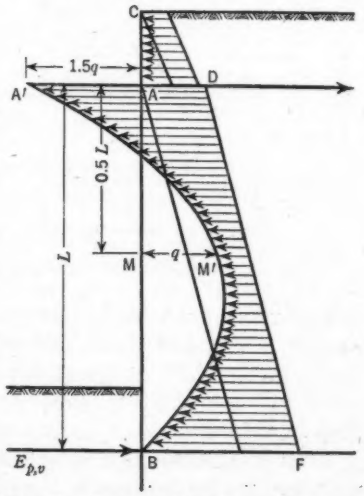

FIG. 22.—PRESSURE DISTRIBUTION RECOMMENDED BY THE DANISH SOCIETY OF ENGINEERS

Fig. 23 shows the limits of the K-value variation along the depth of the bulkhead; and therefore it also shows the limits of the pressure distribution (according to Eq. 4), as established with sand during the Princeton tests. The shape of the Princeton curves is quite different from the shape of the curve corresponding to the Danish recommendations. Again, as in the case of the sand-clay mixture (see Fig. 19), only in the lower zone do the sand tests show a departure from conventionally assumed hydrostatic pressure distribution.

[26] "Normer for Vandbygnings-Konstruktioner," Udgivet af Dansk Ingeniøforening, Køpenhaven, 1937.

D. Restraining Effects at Lower Boundary

On the basis of all the available evidence (subject to further checks) the writer has formed an opinion that the dominant factor in any actually possible reduction of pressures may be the influence of an underlying rigid boundary in relation to the rigidity of the backfill.

Under actual prototype conditions the lower boundary (that is, the dredged line) will be located as shown in Fig. 23 well above the level of the resultant of passive earth pressure (B-level, Fig. 23). Accordingly, the effects of this

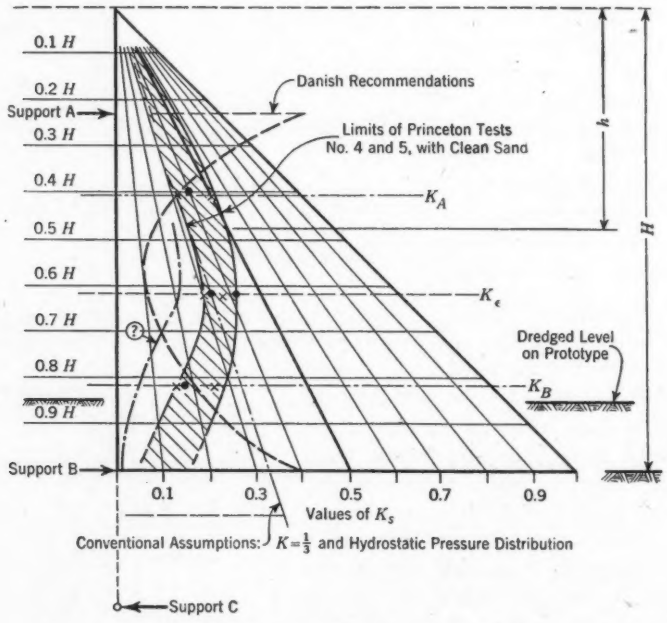

Fig. 23.—Estimated Pressure Distribution of Sand Backfills

boundary should extend further so that a distribution of pressures may result as shown by a dotted line and a question mark in Fig. 23. As in Fig. 19, the curve marked "?" represents an hypothetical pressure distribution that may occur in the field if the relative dredged level in the prototype is higher than that in the model.

The great effect that the nature of a boundary may have on pressure distribution, on its relation to the stiffness of the soil, and on its relation to the intensity of vertical pressures acting on it is illustrated in Fig. 24. Fig 24(*a*) shows the increment of bending strains recorded right after the application of a surcharge of 125 lb per sq ft to a clay backfill consolidated under its own weight. The shears and lateral pressures, computed by the use of Eqs. 1 and 2, as well as the measured increments of pore pressure are also shown. A sandbank, sloping upward and away from the toe of the bulkhead, was in place during that test. As a result of the outward yield of the bulkhead caused by

lateral pressure against the upper three fourths of its height, lateral pressures actually decreased against the lower bulkhead section. This result not only can be noted from the curve of computed pressures in Fig. 24(*a*), but it also follows from the clearly reversed curvature of the directly measured bending strains. Furthermore, the pore pressure cells did not register any increase of pressure in that lower backfill zone. The unconfined compressive strength of that part of the clay backfill at that test stage should have been somewhat higher than the surcharge applied.

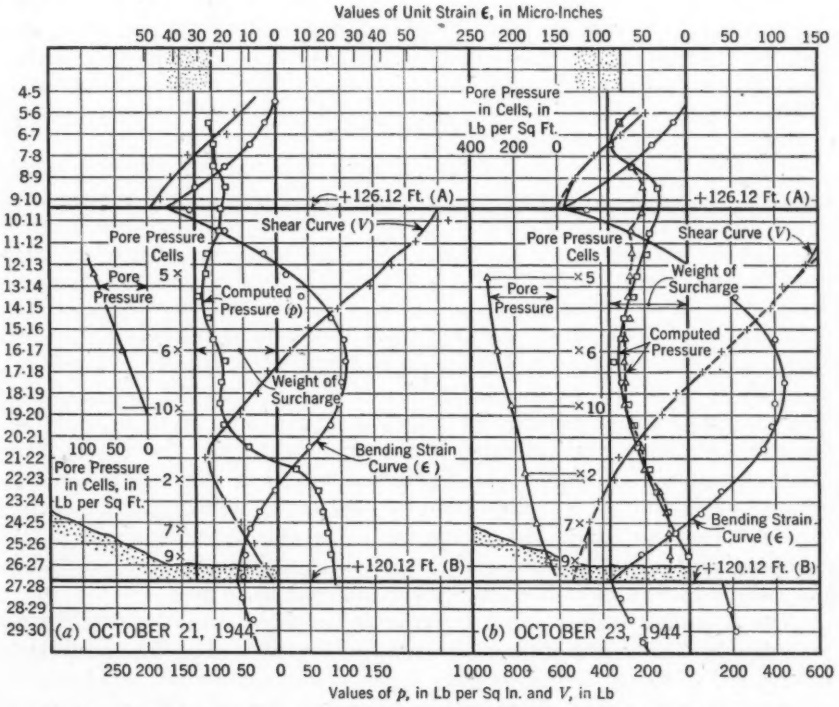

FIG. 24.—INCREMENT OF BENDING STRAINS AND OF PRESSURES AS A RESULT OF SURCHARGE
(*a*) Surcharge, 2 Ft of Water
(*b*) Surcharge, 6 Ft of Water

As the surcharge was increased to 375 lb per sq ft, or to approximately twice the value of the unconfined compressive strength of the clay backfill, a positive increment of lateral pressure and of pore pressure was registered in the lower zone of the backfill (see Fig. 24(*b*)). Nevertheless these pressure increments decreased along the depth of the bulkhead.

In connection with the entire problem of boundary effects, small-scale experiments by Ek-Khoo Tan, Jun. ASCE, showed how the slip plane was affected by the boundary location and how it could develop only at some distance from it.[27]

[27] "Stability of Soil Slopes," by Ek-Khoo Tan, *Proceedings*, ASCE, January, 1947, p. 19.

E. Points to be Considered When Designing Anchored Flexible Bulkheads

It does not appear advisable to rely on the arching effect in sand backfills for a reduction of bending moments. On the contrary, construction procedures should take into account the possibility of local pressure increases resulting from arching in a horizontal direction, unless backfilling is advanced more or less uniformly.

The use of a sand dike offers a practical method of decreasing fluid lateral pressures of unconsolidated backfills with high clay content. In addition, if a rapid increase of the supporting power of such backfills is desired, an attempt can be made to achieve this end by using either conventional type vertical sand drains, or a more effective but more complicated procedure involving an interlaced layered system of horizontal prefabricated drains connected to well points.

If a fluid backfill is used which is composed entirely of a cohesive or a semicohesive soil (such as a sand-clay mixture) and if such a backfill is placed rapidly, then bulkhead design should be based entirely on fluid pressures. If time permits placing such a backfill in layers and accelerating their consolidation by drains of any type, then lateral pressures smaller than fluid ones can be used. However, it does not appear possible to relate the values of these smaller lateral pressures to the shearing strength of the plastic backfills as determined from laboratory tests carried to failure; nor does it appear rational to attempt to do so. Neither the theory of elasticity nor the theory of plasticity is practical for analyzing all phases of the problem. The necessary soil constants cannot yet be evaluated in the laboratory. For that reason it appears advisable to determine lateral pressures for use in bulkhead design from empirical charts of the type illustrated by Figs. 19, 23, and 25, based on actual measurements on models and in the field. Laboratory tests would still be necessary, but would be used mainly for soil classification and identification purposes. Two types of K-coefficients would have to be used.

The K_s-values would be used for granular backfills, since these coefficients are related to the weight of the solids only (see Fig. 25). The K_{s+w}-coefficients are related to the combined weights of solids and water under conditions of complete saturation, and are suggested for use in connection with cohesive and semicohesive soils (see Figs. 19 and 25(*b*)). The two coefficients can be related to each other by the formula:

$$K_s = \frac{K_{s+w} - C'}{1 - C'} \qquad (8)$$

in which

$$C' = \frac{1 + e}{G + e} \qquad (9)$$

In Eq. 9, e is the void ratio; and G is the specific gravity of solids. In Eq. 8, K_s refers to the buoyed weight of the solids and K_{s+w} refers to the nonbuoyed combined weight of soil and water.

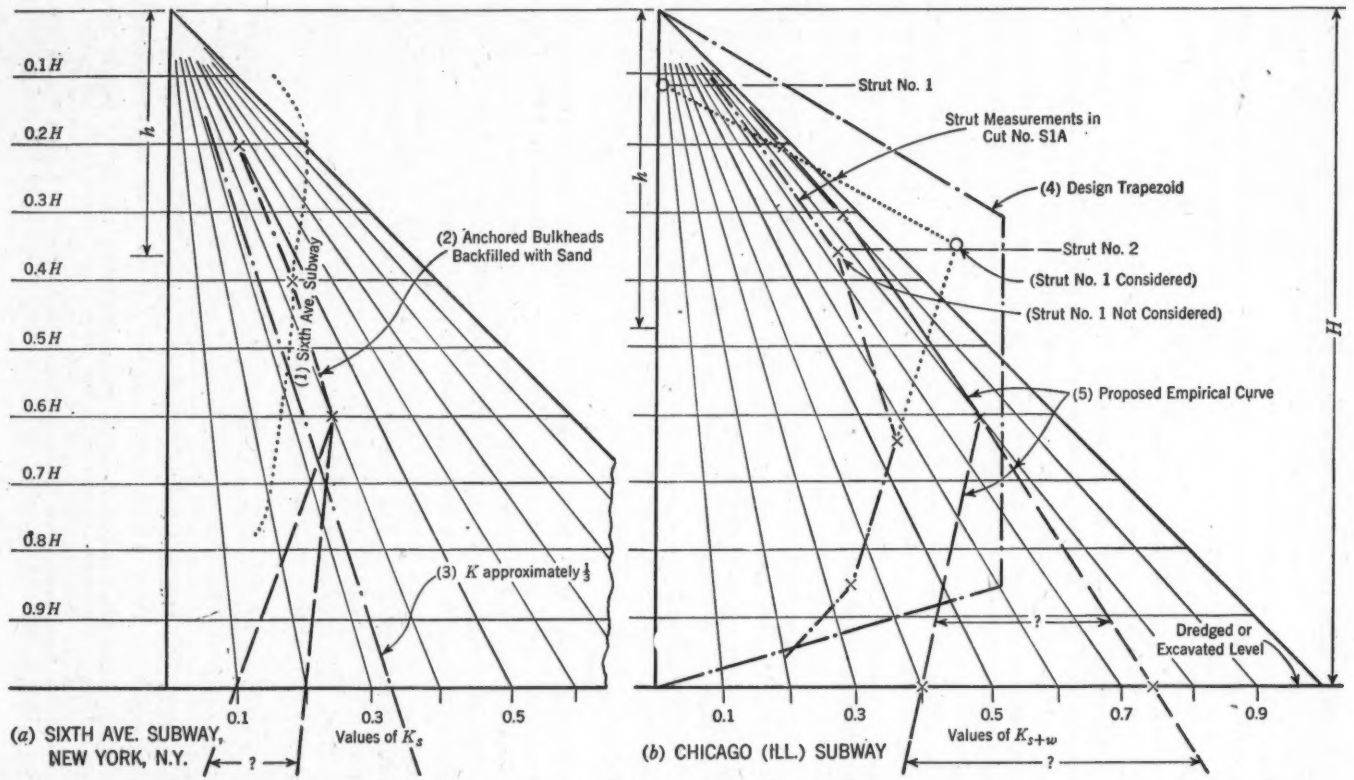

FIG. 25.—OBSERVED PRESSURE DISTRIBUTION IN FULL-SCALE CUTS COMPARED TO DESIGN RECOMMENDATIONS FOR CUTS AND FOR ANCHORED BULKHEADS

Fig. 25(*a*) shows the type of K_s-chart suggested for use with granular backfills and anchored sheet pile bulkheads. The curve obtained from the first series of Princeton bulkhead tests with submerged backfill (curve (2), Fig. 25(*a*)), compared to conventional assumptions, showed a similar but less pronounced trend than did the curve obtained from strut measurements performed in New York on the Sixth Avenue Subway (curve (1), Fig. 25(*a*)).[28] The soil there was wet, but free draining, sand.

In 1941, Professor Terzaghi[29] suggested a method of determining lateral pressures, which method was also used by him and by Professor Peck on the Chicago Subway.[30] It consisted of a pressure trapezoid based on K-values related to laboratory strength tests (curve (4), Fig. 25(*b*)).

The writer's suggested procedure (curve (5), Fig. 25(*b*)) is based instead only on empirical K-values determined from actual measurement. It abandons the trapezoidal shape which, apparently, was recommended mainly for the purpose of providing for possible pressure variations between individual differently wedged braces in cuts.[31] Variations in the sequence and in the timing of the bracing in relation to the progress of the excavation were taken care of by the trapezoidal shape of the pressure diagram, as shown in Fig. 25(*b*), for cut No. S1A.[32] During construction strut No. 1 was placed after the excavation had reached the level of strut No. 2. Similar erratic variations cannot occur in the case of an anchored bulkhead, and the writer's suggested K-values or pressure variation curves are triangular in the upper one half to two thirds of the height of the bulkhead, with a parabolic decrease thereafter.

Fig. 25(*b*) gives a K_{s+w}-chart proposed for use with plastic cohesive and semicohesive soils—that is, with clays and with mixtures of sand and clay. The term "plastic" refers to soils that require approximately 20% strain in order to fail when subjected to an unconfined compressive strength test. A curve of the K-values or pressure variation measured on the Chicago Subway and computed by the writer from strut loads given by Professor Peck[32] for cut No. S1A (curve (5), Fig. 25(*b*)) has a shape similar to the one determined from the Princeton model tests with consolidated fluid backfills, except for a more pronounced effect of the lower boundary, in accordance with the semirigid structure of the natural deposit of Chicago clay. The lateral pressures are also correspondingly smaller. The corresponding "design trapezoid" recommended for use on the Chicago Subway cuts by Professor Peck[30] is shown as curve (4), Fig. 25(*b*). Its application to anchored bulkhead problems would produce much too high anchor pull values, as compared to the method proposed by the writer.

Fig. 25 is intended to illustrate only the method proposed. The values on these diagrams are based on the first series of tests only. They are conserva-

[28] "Cofferdams," by Lazarus White and Edmund Astley Prentis, Columbia Univ. Press, New York, N. Y., 1940, p. 79.

[29] "General Wedge Theory of Earth Pressure," by Karl Terzaghi, *Transactions*, ASCE, Vol. 106, 1941, pp. 77–78.

[30] "Earth-Pressure Measurements in Open Cuts, Chicago (Ill.) Subway," by Ralph B. Peck, in "Earth Pressure and Shearing Resistance of Plastic Clay: A Symposium," *ibid.*, Vol. 108, 1943, p. 1032.

[31] "General Wedge Theory of Earth Pressure," by Karl Terzaghi, *ibid.*, Vol. 106, 1941, p. 77.

[32] "Earth-Pressure Measurements in Open Cuts, Chicago (Ill.) Subway," by Ralph B. Peck, in "Earth Pressure and Shearing Resistance of Plastic Clay: A Symposium," *ibid.*, Vol. 108, 1943, p. 1027, Table 3.

tive and are subject to revision, especially in the lower backfill zones, as further test and observation data become available.

PART III. THE SECOND AND THE THIRD SERIES OF TESTS

A. Points that Required Further Clarification After Completion of First Series of Tests

The testing setup described in Part I permitted the controlled motion of the model bulkhead supports and the study of the effects of various types of such motion on lateral pressures. A separate study of this variable would not have been possible with a model bulkhead actually embedded in soil.

The results described in Part II clarified a number of important points, especially in respect to lateral pressures in the upper backfill zone and to anchor pulls. The model bulkhead was designed and was operated to have full model similarity with prototypes in respect to its flexibility. The similarity of behavior of noncohesive soils in models and in prototypes is generally known.[6,29]

However, as explained in Part II, there remained some questions concerning the validity of the results obtained in the lower backfill zones. Specifically, these questions were related to:

(1) The effect on backfill pressures and on bulkhead and on anchor stresses of the rigidity of soils underlying the dredged level;
(2) The values of lateral pressures of cohesive soils exerted under prototype intensities of vertical pressure;
(3) The performance of soils other than the types tested; and
(4) The effect of wall rigidity, as compared to the "spring action" of deflected bulkheads, on the lateral pressures of consolidating clay backfills.

Two additional series of tests were initiated for the purpose of clarifying these four points.

The second series of tests was started with the "lateral pressure meter." This apparatus is illustrated subsequently in Fig. 29 and is described in the third paper of the Symposium. This second series of tests is intended to clarify questions (2), (3), and (4).

A third series of tests was started for the purpose of clarifying question (1). A new model bulkhead setup for combined active and passive earth pressure tests was used. It is described in the third paper of the Symposium.

B. First Results of Second Series of Tests

Several tests have been performed with the "lateral pressure meter" (illustrated subsequently in Fig. 29) under intensities of vertical pressure equal to 0.25 ton per sq ft, 0.50 ton per sq ft, and 1.00 ton per sq ft. These vertical pressures correspond to the buoyed weight of approximately 8 ft, 17 ft, and 33 ft of submerged soil.

Originally fluid red clay (see Fig. 9) was used at a consistency of approximately three blows on the liquid limit device. In the upper unrestrained zone of the apparatus the initial ratio of lateral to vertical pressure was found to be approximately $K = 1.00$. After completed consolidation this value decreased

to approximately $K_s = 0.50$ for all three previously mentioned vertical pressure intensities. According to Eq. 8 the corresponding value for a nonbuoyed but saturated soil should be $K_{s+w} = 0.75$. During the first series of model bulk-

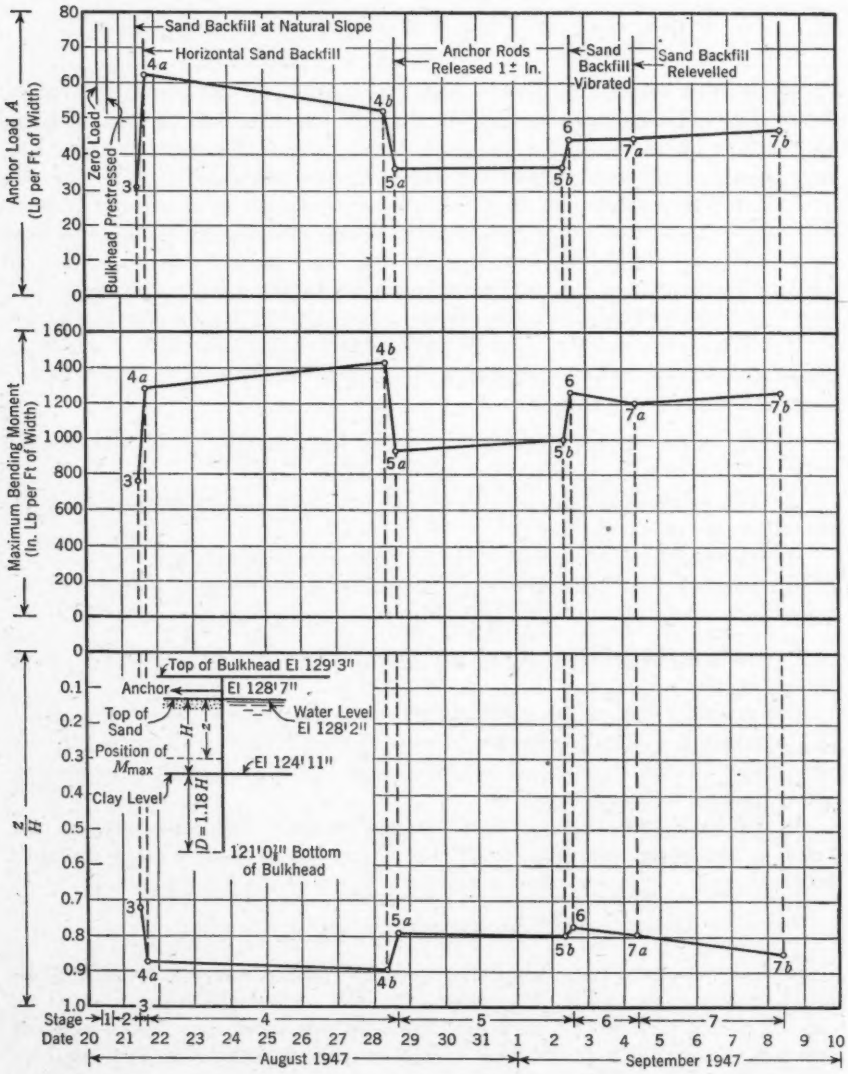

FIG. 26.—RESULTS OF FIRST TEST WITH NEW MODEL BULKHEAD SETUP FOR COMBINED ACTIVE AND PASSIVE EARTH PRESSURE TESTS (TEST 11A)

head tests this value was found to lie between 0.70 and 0.80. The similarity between model and prototype can therefore be considered proved in respect to question (2), Section A, Part III.

These tests were performed with almost perfectly rigid supports, the maximum yield of which was approximately 0.003 in. (or 0.00025 of the sample diameter). The elastic recovery of the supports after completed consolidation was approximately one third of the foregoing values. It would appear, therefore, that question (4), Section A, Part III, can be answered in the sense that the wall rigidity has little effect on the decrease of lateral pressure during the consolidation of the clay backfills.

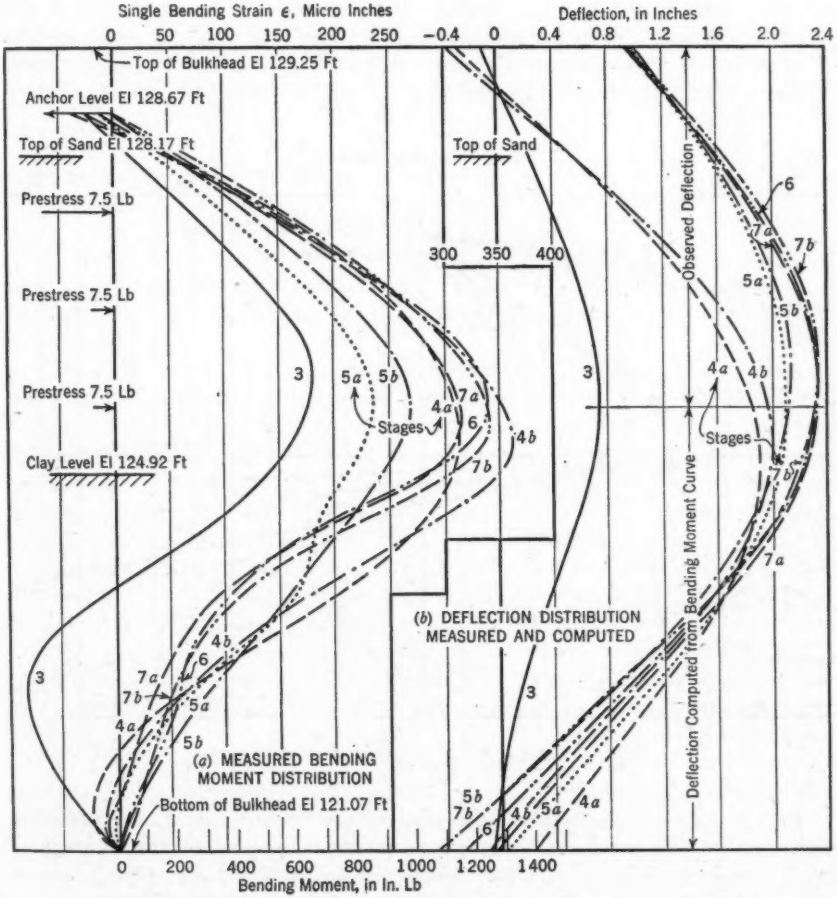
Single Bending Strain ϵ, Micro Inches
0 50 100 150 200 250
Deflection, in Inches
−0.4 0 0.4 0.8 1.2 1.6 2.0 2.4
Top of Bulkhead El 129.25 Ft
Anchor Level El 128.67 Ft
Top of Sand El 128.17 Ft
Top of Sand
Prestress 7.5 Lb
Prestress 7.5 Lb
Prestress 7.5 Lb
Clay Level El 124.92 Ft
Observed Deflection
Deflection Computed from Bending Moment Curve
Stages
Stages
(a) MEASURED BENDING MOMENT DISTRIBUTION
(b) DEFLECTION DISTRIBUTION MEASURED AND COMPUTED
Bottom of Bulkhead El 121.07 Ft
0 200 400 600 800 1000 1200 1400
Bending Moment, in In. Lb

FIG. 27.—VARIATION IN TEST RESULTS FOR DIFFERENT TEST STAGES MEASURED

After completed consolidation a release of the ring supports equal to 0.3 in. or to 2.5% of the sample diameter did not decrease the lateral pressures exerted against the upper rings any further.

These findings confirm the corresponding conclusions reported under Section B, Part II.

C. First Results of Third Series of Tests

The new setup for combined active and passive earth pressure tests (see the third paper of the Symposium) has been carefully calibrated and was found to work satisfactorily. The results of the first test performed with it are shown by Figs. 26, 27, and 28.

The model bulkheads were embedded to a depth of 46 in. in softly plastic red clay (Fig. 9) with a consistency of thirty-nine blows on the liquid limit device. This consistency corresponds to an unconfined compressive strength of approximately 300 lb per sq ft in the material. A depth of sand backfill $H = 39$ in. was then placed up to the water level 5 in. below the anchor rods. The average dry density of the sand after the end of the test was 93 lb per cu ft.

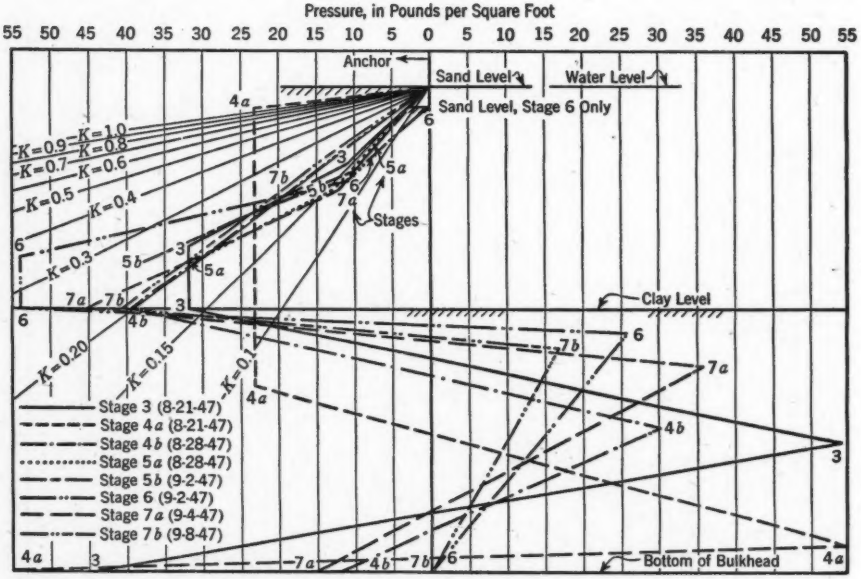

Fig. 28.—Lateral Pressure Distribution (Computed)

Fig. 26 shows the variation with time of the anchor pull A, the variation with time of the maximum bending moment M_{max}, and of the location of M_{max} in terms of the ratio z/H. All recorded variations are entirely logical and are in agreement with each other and with the testing sequence. It can be seen that the location of the maximum bending moment is just as important as the distribution of active pressures.

The variation for different test stages of the bending moment distribution is shown by Fig. 27(*a*); of the deflection distribution by Fig. 27(*b*); and of the estimated pressure distribution by Fig. 28. The progress of the test, recorded by stages, is as follows:

Stage	Date (1947)	Description
3	August 21	Backfill completed on natural slope
4*a*	August 21	Backfill leveled off
4*b*	August 28	Same condition, 7 days later
5*a*	August 28	Anchors released 1 in.
5*b*	September 2	Same condition, 5 days later
6	September 2	Sand backfill compacted by vibration
7*a*	September 4	Sand added to previous level
7*b*	September 8	Same condition 4 days later

The unit pressures were first computed down to the depth z corresponding to the location of the measured maximum bending moment and hence of zero shear. Their distribution to that level was estimated from the two equations:

$$A = \Sigma(p_h\, dz) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10a)$$

and

$$A\,(z + 5 \text{ in.}) - (\Sigma p_h\, dz)\, z_o = M_{max} \quad \ldots\ldots\ldots\ldots (10b)$$

This distribution was then similarly checked by means of the measured bending moment at the depth $z/2$ and by means of the procedure governed by Eqs. 1 and 2. The pressure distribution below depth z was estimated in a similar manner.

Test stages 4*b* and 7*b* represent relatively stable conditions and therefore appear most suited for consideration as a basis for design recommendations. Such recommendations can be made in final form only after completion of the entire test program. At present, however, it can be stated that the knowledge of three factors is sufficient for the design of an anchored bulkhead. These three factors are: (1) The minimum safe depth of embedment of the bulkhead; (2) the anchor pull value; and (3) the value of the maximum bending moment.

The first factor, as well as the possible variations of the value z, is the subject of further studies within the third series of tests. Once a safe depth of embedment is selected for a given set of conditions, the knowledge of the depth z at which M_{max} is located below the backfill surface in conjunction with active earth pressure diagrams of the type shown by Figs. 19, 23, 25(*a*), 25(*b*), and 28 permits the rapid determination of the anchor pull A and of the maximum bending moment. Fig. 28 shows that for the test stages 4*b* and 7*b* the active K_s-value does not exceed 0.22. This substantiates the earlier statement that the tentative recommendations shown by Fig. 25(*a*) are of a conservative nature, as was intended prior to the completion of the test program.

Acknowledgments

As stated in the foreword by Admiral Smith, the project reported in this paper was sponsored by the Bureau of Yards and Docks of the United States Navy, under Rear-Admiral J. J. Manning, M. ASCE, as chief of the Bureau. Commander L. C. Coxe, as officer in charge of the project for the Navy, devoted considerable time and effort to the organization of this Symposium. Admiral Ben Moreell, Hon. M. ASCE, and Rear-Admirals L. B. Combs and W. H. Smith, and Capt. E. H. Praeger, Members, ASCE, were instrumental

in initiating the project. Lieutenants S. C. Nickerson and R. Z. T. Anthony also took an active part in the early planning stages of the project. Harris Epstein and L. A. Palmer participated in discussions of the test results.

At Princeton University it was Philip Kissam, M. ASCE, who initiated the project. The writer, as engineer in charge of the project, is indebted to Elmer K. Timby, M. ASCE, chairman of the Department of Civil Engineering, and to Kenneth H. Condit, dean of the School of Engineering, for their continued support. Edwin L. Kimble, and Ek-Khoo Tan, as research associates, helped the writer greatly through the early and most difficult stages of its development. The latter, with Harris Epstein, should receive credit for suggesting the sand dike between the bulkhead and the fluid clay backfill. Students, under the supervision of Professor Timby, performed check computations of deflections from measured bending moments. For providing the necessary space, which resulted in an appreciable reduction in cost of the installation, the project is indebted to the Princeton Athletic Association.

The entire design was made at Princeton; and the writer is particularly indebted to N. J. Sollenberger and David H. Crater, Assoc. Members, ASCE, for help in this connection. A total of thirty-six design drawings were prepared for official Navy approval.

The authors of the third paper of the Symposium, as well as J. D. Welch, performed the tests which provided the data reported in Part III.

The mudjacks mentioned in Part I, Section C, were rented from the New Jersey State Highway Department through an arrangement authorized by the courtesy of Spencer Miller, State Highway Commissioner. Other members of the department—Alex W. Muir, F. D. Woodruff, M. A. Lanzara, and C. H. Weller—were materially helpful in this phase of the work.

Roy W. Carlson, Assoc. M. ASCE, designed for the project specially sensitive pore pressure cells which were used to measure the variation of neutral stresses within the backfill in which they were embedded.

Over all, the writer is indebted to the friendly interest of Professor Terzaghi, in acquainting the writer with the potentialities of electric resistivity gages before they became readily available.[7]

SPECIAL FEATURES ON LARGE-SCALE EARTH PRESSURE TESTS

BY EDWARD R. WARD,[33] ASSOC. M. ASCE, JOHN R. BAYLISS,[34] ESQ., AND PHILIP P. BROWN,[35] JUN. ASCE

INTRODUCTION

The second and the third series of the Princeton tests outlined in Part III of Professor Tschebotarioff's paper required the development of new equipment, the special features of which are described in this paper.

PART I. THE LATERAL EARTH PRESSURE METER

The second series of the Princeton tests is mainly concerned with the determination of lateral pressures under vertical pressure intensities corresponding to values reached on the prototype. The comparative effects of immovable and of elastically and plastically yielding supports on changes of lateral pressure of different types of soils represent another important question studied. In an attempt to isolate and study some of these factors individually, it was decided to construct a device which has been called a "lateral pressure meter." This device is capable of testing a cylinder of soil 1 ft in diameter and 1.5 ft high, with a normal load at the top, and of measuring lateral pressures and lateral displacements at small increments throughout its depth.

Its general shape (see Fig. 29) is that of a thick-walled cylinder cut in half vertically. The left half (M) is solid from top to bottom; but the right half is sliced horizontally into twelve half-circular rings (N), Fig. 29. These rings are mounted on ball bearings so that each one is free to roll out away from the solid half. The end of each ring is connected to the solid half by a dynamometer (R) on which electric strain gages (S) are mounted, thus permitting the total lateral pressure on any ring to be determined. Outward motions of the rings are measured by mechanical dials (V).

Normal vertical pressure is applied to the top of the soil sample by air pressure at point E which acts through a thin rubber membrane (B) that is free to adjust its shape to conform to the contour of the soil surface. This procedure insures that no shearing forces can be transferred to the upper boundary of the soil surface when the vertical pressure is applied. Vertical settlement is measured by a series of indicator rods (F), resting on the rubber membrane in contact with the soil surface, which activate the mechanical dials.

The entire device is surrounded by a watertight jacket (U), Fig. 29, permitting tests on submerged soils. The inner faces of the soil chamber are lined with porous stone (X) to permit lateral drainage.

At the start of a test the soil sample (A) may be consolidated for a given length of time at any desired pressure up to 3 tons per sq ft. During this period

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of consolidation the variations in lateral pressure at different depths may be noted from the change in the dynamometer readings. The twelve movable rings (N) may be supported in at least three different ways, depending on how the dynamometers holding them are fastened onto the fixed half cylinder: (*a*)

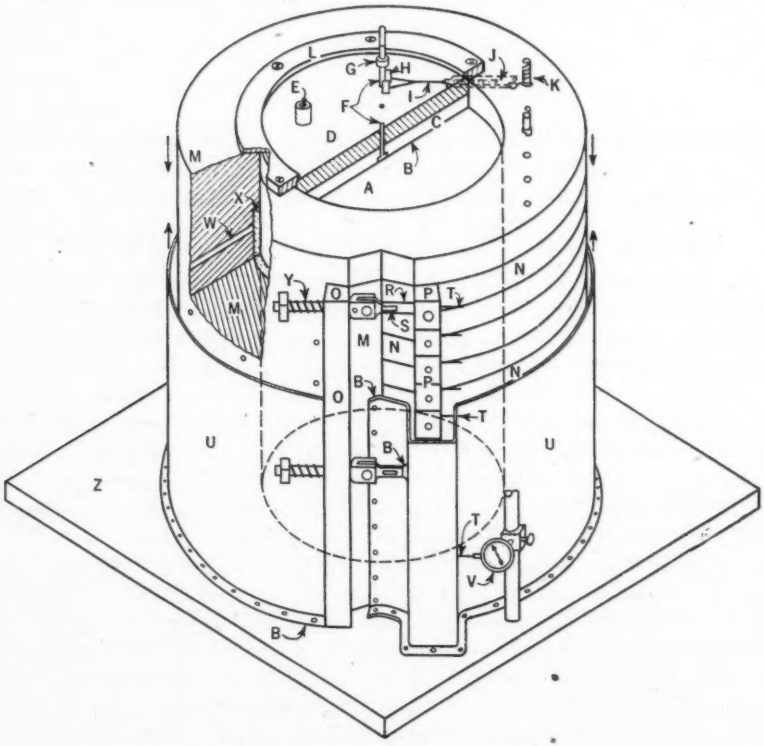

FIG. 29.—LATERAL EARTH PRESSURE METER

Rigidly—bolted down; (*b*) elastically—pulling against a coiled spring (Y); or (*c*) in such a manner that a controlled release is permitted to remain as permanent "set."

The laboratory of the United States Engineer Department at Seattle, Wash., uses a device somewhat similar to Fig. 29 for measuring the lateral pressures of coarsely graded materials only. This machine, however, uses a rigid plunger to apply the vertical load, and the lateral deformations and pressures are measured together as a unit when the observer notes the deformation of the calibrated springs holding its five movable rings in place.

Quite a number of variable testing procedures are afforded by the device described in this paper, depending on the soil used, extent of consolidation, applied pressure, method of supporting the rings, and other factors, for which provisions have been made in the design, such as pore pressure measure-

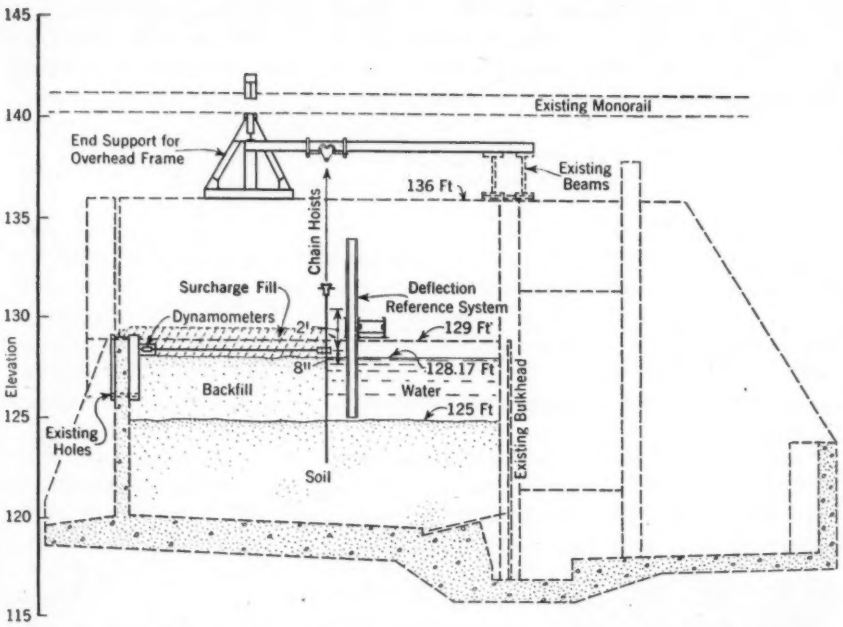

FIG. 30.—MODEL BULKHEAD FOR COMBINED ACTIVE AND PASSIVE EARTH PRESSURE TESTS

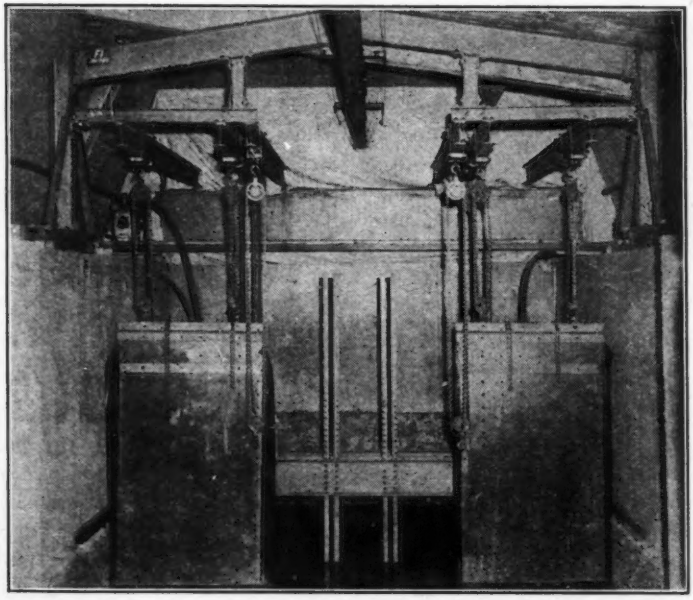

FIG. 31.—VIEW OF THE TWO SIDE BULKHEADS DURING CONSTRUCTION, DECEMBER, 1946

ments and electroosmotic phenomena. Initial preliminary calibrations were completed satisfactorily, using a rubber bag filled with air under pressure in place of soil. Results obtained from tests with actual soil are given in Section B, Part III, of the second Symposium paper.

PART II. COMBINED ACTIVE AND PASSIVE EARTH PRESSURE TESTS

A. General Layout.—The third series of the Princeton tests was concerned with the model study of flexible bulkheads embedded in soil at their lower end. A setup for combined active and passive earth pressure tests was designed for the purpose and is illustrated by Fig. 30. Fig. 31, a view taken from the anchorage end of the bulkhead during construction, shows two side bulkhead sections. At this stage, the center bulkhead section (on which reaction, bending strain, and deflection measurements are taken) had not been placed in position.

The overhead steel structure and chain hoists permit the adjustment of the bulkhead in vertical and longitudinal positions for various test conditions. The vertical channels attached to the rigid horizontal frame shown in the center of Fig. 31 are provided as a support for gages designed to measure the deflection of the center bulkhead. The direct measurement of the deflection of the bulkhead is desired as a check on deflections computed from the bending strain measurements, the latter being obtained by means of electric resistivity strain gages attached to the center bulkhead and waterproofed.

FIG. 32.—VIEW OF THE CENTER BULKHEAD DURING MOUNTING OF THE GAGES, DECEMBER, 1946

B. Underwater Measurements of Bending Strains.—In order to test the model bulkhead for combined active and passive earth pressure, some effective method had to be devised to waterproof the electric resistivity strain gages for use under water. The method, as originally[36] developed, consisted in baking the gages on the plate until proper electrical resistance between gage and plate was obtained. Then a sheet of "Koroseal" was applied over the gages and cemented firmly to the plate with Koroseal cement. Hot vaseline was poured into the semicylindrical space between the gages and the Koroseal.

In Fig. 32, the model bulkhead (5 ft wide) is held in an upright position by a wooden frame. On the right-hand side is an uncovered strip of several gages which have not been waterproofed. The strip on the left shows the manner in which the lead wires were arranged to facilitate application of the Koroseal. At the bottom the Koroseal strip is cemented over the gages. Into this space between the flexible rubber sealing strip and the plate, hot

[36] "A Method of Effecting SR-4 Strain Gage Operation Under Water," by Edwin L. Kimble, *Proceedings*, Soc. for Experimental Stress Analysis, 1946, Vol. III, No. 2, pp. 53–54.

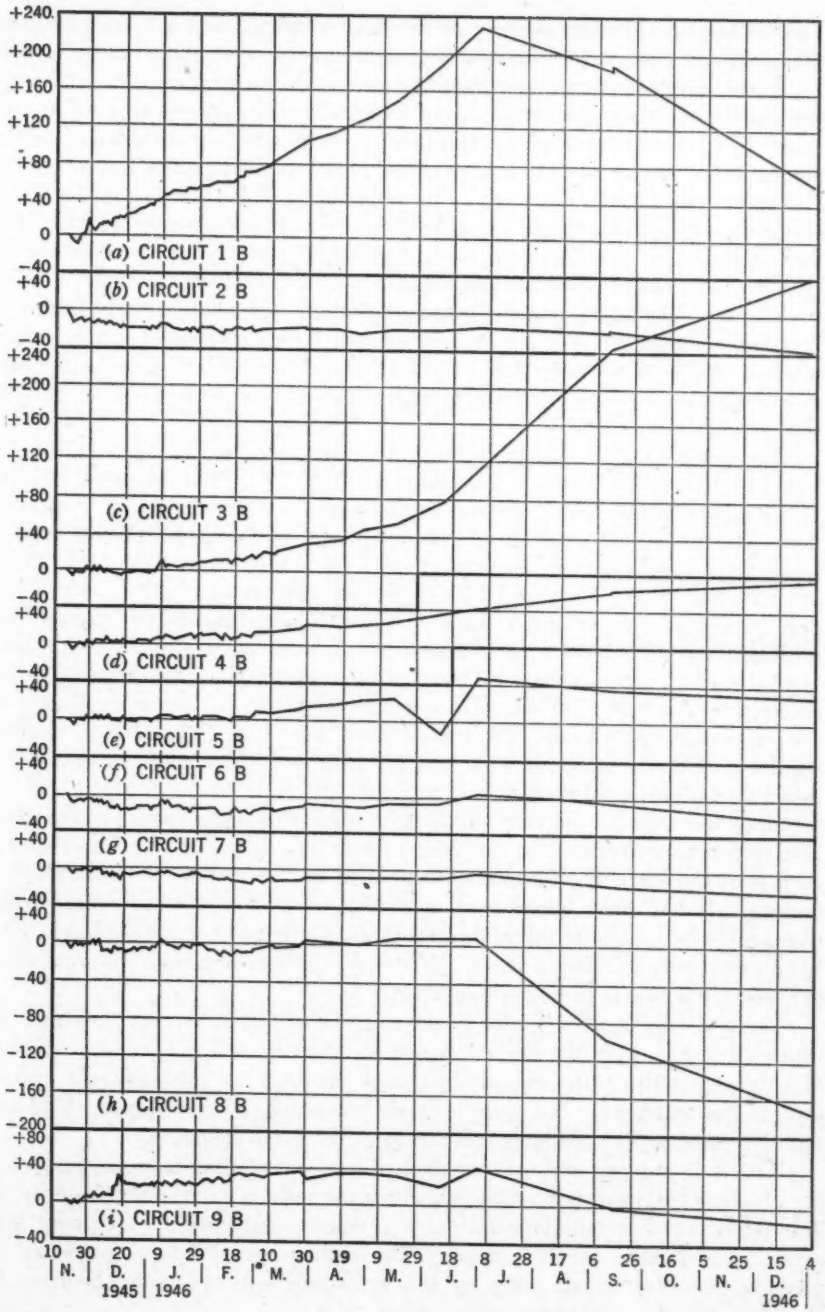

FIG. 33.—RECORD OF SUBMERGED STRAIN GAGE READINGS

vaseline is poured in lifts sufficiently small so as not to burst the bond between Koroseal and plate with hydrostatic pressure.

In the center there is a completed strip—the Koroseal having been covered with a copper strip merely for protection. Two strips are applied at one time—one on either side of the bulkhead. By proceeding in this manner the copper strips can be bolted on immediately after cementing the Koroseal strip and before pouring the vaseline. The copper gives added support to the Koroseal strip and permits pouring molten vaseline in lifts of greater depth. The arched section of the copper is slotted at 1.5-in. intervals to prevent its increasing the rigidity of the bulkhead.

The stability tests of gages waterproofed in this manner were begun in November, 1945. The gages were cemented to an unstressed plate, waterproofed, and then submerged in a tank of water. Direct and reverse readings were taken throughout to eliminate any error resulting from a "zero shift" of the reading instrument.

The behavior of nine gage circuits throughout the period from November, 1945, to December, 1946, is illustrated by Fig. 33. The ordinates indicate variations from the initial readings of the gages. Over a period of 6 months from November, 1945, to May, 1946, only two of the gage readings varied more than approximately 20 micro-inches from the initial readings. These two circuits (Figs. 33(*a*) and 33(*c*)) showed general instability. Since the observers were able to obtain an accuracy of approximately 10 micro-inches during all previous tests with this instrument, after discarding approximately one third of all gage readings subsequent to recalibration, this variation is not excessive. However, beyond this 6-month period three more gage circuits showed variations considerably greater than would be acceptable as reliable readings. Consequently, beyond the 6-month duration, readings cannot be relied on without intermediate calibrations.

Since the tests on the new bulkhead, designed for active and passive pressure research, are planned to run over a maximum period of 2 or 3 months for each test, the stability of gages waterproofed in this manner is entirely satisfactory. It had been hoped that, with this method of waterproofing, strain gages could be applied to full-scale structures in order that strain readings could be recorded over a period of years. However, further research in waterproofing is necessary if this is to be accomplished.

Three independent strips of gages have been placed on the bulkhead at intervals of 2 in. over the lower 3 ft and at intervals of 3 in. over the upper 4 ft. Consequently, three independent gage readings are made at each elevation on the bulkhead. If a gage becomes unreliable, for any reason, there are still two independent readings to average at the same elevation.

With this number and arrangement of gages a bending-moment curve for the stressed bulkhead is possible which occasionally is sufficiently accurate to determine, by direct differentiation, a pressure distribution curve over the active and passive pressure areas. The estimation of lateral pressures is possible, however, in all cases by the procedure outlined under Section C, Part III, of the second Symposium paper.

C. Underwater Deflection Measurements.—To measure deflections it was first found necessary to solve the problem of making underwater measurements with a desired accuracy of 0.001 in. and a range of 1½ in. After much investigation, it was decided that this could be accomplished by the use of the linear variable differential transformer shown in Fig. 34. This is a sketch of the

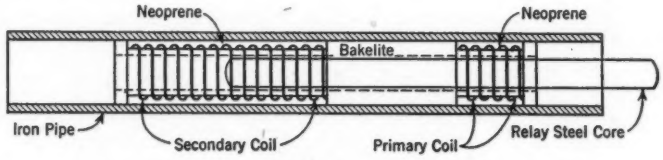

FIG. 34.—LINEAR VARIABLE DIFFERENTIAL TRANSFORMER

transformer deflection gage as built to meet the requirements of the test. It consists of a primary coil 1 in. long wound on a bakelite spool approximately ¾ in. in diameter and carries a constant voltage. On the same spool 2 in. away a secondary coil, 3 in. long, is wound. The voltage of this coil is determined by the position of a relay steel core (6½ in. long by $\frac{7}{16}$ in. in diameter) as it moves along the axis of the coils. The change in voltage created by the change in position of the core may be measured with a highly sensitive voltmeter. Then, using a previous calibration of the voltage readings against the movement of the core, the extent of this movement can be determined at any time by reading the change of voltage.

To ascertain its stability under water, a transformer gage, with its core fixed in a definite position, was submerged in water for about 3 months, no precautions being taken to waterproof it. Periodic readings of the voltage across the secondary coil were taken and were found to vary within about 3%, ignoring errors from power line voltage fluctuations. As an added precaution, however, in the construction of the gages for use on the bulkhead, the coils were waterproofed with "neoprene." The steel cores also were painted with neoprene to prevent them from rusting.

It was decided to mount six of these gages on the vertical channels shown in Fig. 31. These channels serve as fixed reference supports. The gages are placed at three elevations below water level at points where deflection readings are desired, with two gages 20 in. apart at each elevation to measure any twisting of the bulkhead. The cores of these gages are attached to the center bulkhead as shown in Fig. 35.

In addition to the electrical measurement, it was thought advisable to provide some method of a fairly accurate mechanical measurement of the same points both as a check and as a precaution for some unforeseen failure of the transformer gage. A wire, 0.01 in. in diameter, is attached to the end of the core of the deflection gage away from the bulkhead, passing under a pulley, up through the water into the air, over a second pulley and down to, and attached to, the top of a mechanical dial gage graduated to 0.001 in. The spindle of the dial is weighted with a 4-lb weight to keep the wire taut. Any movement of the core of the gage is transferred to the mechanical dial gage where it can be

easily read. This procedure also permits the measurement of deflections in excess of the range provided for by the transformer gages.

Instead of using a voltmeter calibrated against the movement of the core of the transformer gage to read deflections, it was thought advisable to balance a seventh gage at point A, Fig. 36, using a null balance indicator (C) against each of the gages (B) attached to the bulkhead, and a method was provided to read the deflections directly. By exercising care in manufacture, seven gages were produced which, when tested, showed a total error of only 0.003 in. in the entire range of 1.5 in. The manufacturer built these gages so that they all have the same electrical characteristics in order that an equal movement of the cores (D) in any of the gages gives the same voltage change in any of the secondary coils.

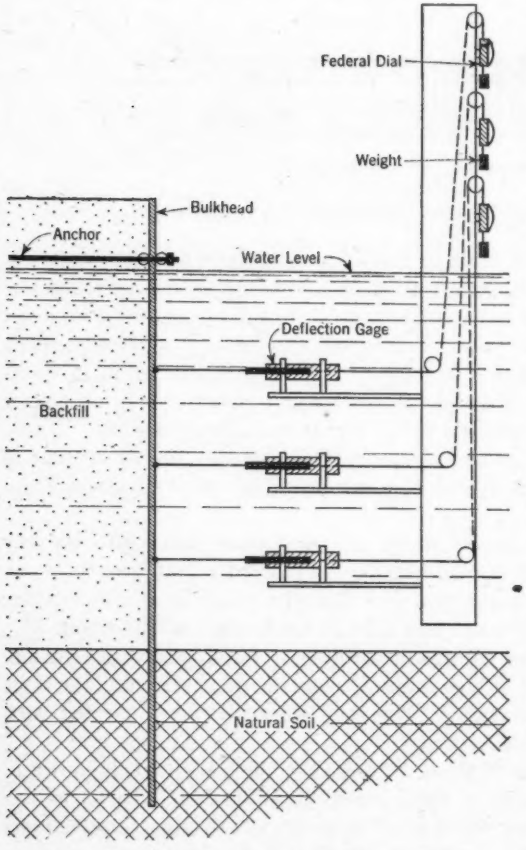

FIG. 35.—UNDERWATER DEFLECTION MEASUREMENTS

A micrometer screw (E) was attached to one of the gages and was mounted on the null balance indicator. With a selector switch (F), Fig. 36, this gage can be connected electrically in a circuit with any of the other six gages which are attached to the bulkhead at points where the deflections are measured.

The null balance indicator was placed in the circuit in such a way that, if there is any movement of the core of the gage at the bulkhead, the pointer on the indicator moves off the zero point. The core of the instrument gage is then moved in or out until the indicator again reads zero. The difference in the readings on the micrometer screw before and after moving equals the movement of the core of the bulkhead gage. The circuit was designed so that any change of voltage in the primary coil from 95 v to 135 v, well within the possible voltage fluctuation of the power supply, has no effect on the relative movement of the cores.

Since it was also found necessary to heat up the gages before readings are taken, a second selector switch (G), Fig. 36, was placed in the circuit, thus permitting all the gages to be heated while recordings are being made. With this transformer gage, a 0.0001-in. movement of the core of the bulkhead gage can easily be detected, with an accuracy of 0.001 in.

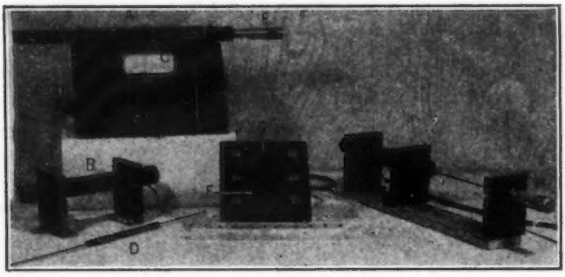

FIG. 36

Because it was desirable to have some method of checking the instrument, a second micrometer screw (H) was provided that can be attached in turn to any of the bulkhead gages. The core of the bulkhead gage can thus be moved in or out any desired distance, and the instrument can then be balanced with the instrument micrometer screw. The movement of the instrument screw is read; and, if any discrepancy exists between this reading and that of the movement of the core of the bulkhead gage, an adjustment can be made with a potentiometer shunted across the primary coil of the bulkhead gage.

Preliminary tests with these gages proved satisfactory. However, during the first bulkhead tests, displacements were found to exceed the range of the gages (see Figs. 26, 27, and 28) so that the mechanical deflection measuring device had to be relied upon.

ACKNOWLEDGMENT

The setup illustrated by Fig. 30 became possible as the result of a procedure developed by Edwin L. Kimble,[36] which permitted the use of the electric resistivity strain gages under water. Professor Tschebotarioff adapted this procedure to the model testing problems at Princeton. The conception and program of study with the lateral pressure meter were developed by Professor Tschebotarioff. C. E. Kjetsaa, instructor in mechanical engineering, supervised machine work and made many helpful design suggestions.

APPLICATION OF TEST RESULTS TO QUAY WALL DESIGN

BY HARRIS EPSTEIN,[37] ESQ.

ECONOMIC CONSIDERATIONS

The problem of reducing the cost of building quay walls and bulkheads is quite old and antedates the science of soil mechanics. Even in "prehistoric" times—when no great distinction was made between clays and sand, when clays were considered ideal foundation materials, before any such thing as pore pressure had been discovered to plague the conscience of economically minded designers, when the subway walls of all the subway systems in New York, N. Y., and Philadelphia, Pa., were designed for a flat K-value of 1/3—even in those distant days the Bureau of Yards and Docks of the United States Navy Department had been grappling with the problem of the excessive cost of quay walls and how to reduce them. The main cost of quay wall construction is invested in two items—(1) the cost of the sheet piling, and (2) the cost of the backfill. To be significant, any saving must be effected in the cost of these two items. The better the backfill material, the shorter will be the length, and the lighter the weight, of sheet piles required for a given dredged depth, and the less will be the cost.

Good filling material, such as sand and other noncohesive soil, is not always available at the site where it can be pumped in hydraulically at a relatively small cost. When good filling material must be brought in from a distance and handled by barges, its cost is greatly increased and, if the quantity of the backfill so handled is large, its increase in cost will more than offset any saving that might be effected in the sheet piling. The same applies to the case in which the original ground underlying the quay wall site is considerably above the required dredged depth. When the original material into which the sheet piles are driven is more or less of a soft, cohesive, unconsolidated nature, the active pressure on the wall becomes quite large especially under the surcharge of heavy live loads. At the same time the resisting passive pressure below the dredged bottom is greatly reduced—resulting in greatly increased length and weight of the sheet piling with considerably increased cost of the quay wall. In fact, where the dredged depth is greater than 25 ft, the moment caused by pressures of unconsolidated backfill is increased to such a magnitude as to preclude the use of commercially rolled steel pile sections. A conventional installation, utilizing the actual materials found in place, is shown in Fig. 37. Again, the evident remedy is to dredge off the soft material and substitute a good granular fill. In this case, however, the cost of filling must also include the cost of dredging and disposing of the original soft material, and this must be balanced against the saving effected in the steel sheet piling.

The unit cost and the quantity of the required borrowed backfill are the two factors that determine whether the use of this borrowed fill construction

[37] Head Designing Engr., Waterfront Structures, Bureau of Yards & Docks, U. S. Navy Dept., Washington, D. C.

method is economical. It is not necessary, however, to have the entire fill made of imported granular material; enough good material may be placed to form a sand dike behind the wall, and the remainder of the backfill may be pumped in hydraulically from the local inferior material.

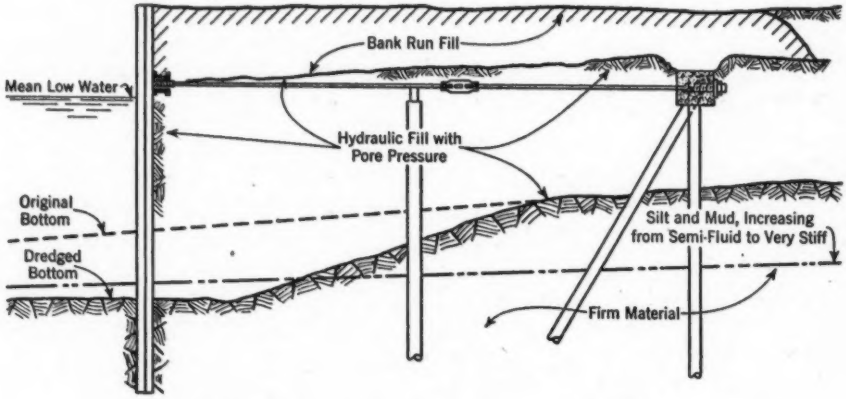

FIG. 37.—CONVENTIONAL CONSTRUCTION OF QUAY WALL

CONSTRUCTION METHODS

The Bureau of Yards and Docks has made use of the borrowed fill method of quay wall construction in a number of instances. Representative examples are the quay walls at Key West and Jacksonville, Fla. At Key West, a quay wall of steel sheet piling (MZ32) was designed for a 22-ft dredged depth to take an hydraulically pumped fill behind a light coral rock dike. At Jacksonville, the use of a relieving platform and a dike of oyster shells made it possible for a steel sheet pile wall (MZ38) to resist 48.0 ft of hydraulically pumped fill of local fine sand. In these cases the relief in lateral pressures is attributed to the high angle of repose and the light weight of the dike materials.

In adopting this construction procedure the reasoning was based mainly on common sense analogy with earth dams, where an earth fill is depended on to keep back the full water pressure. In the case of a quay wall, the pressure on the wall is considerably less than that on a dam of comparable height, since the water pressure acts on both sides of the wall and, therefore, is equalized. The natural slope of the downstream side of the earth dam is replaced in the quay wall construction by the sheet pile wall. The width of the top berm in earth dams is determined primarily by requirements of imperviousness to water seepage and practical construction considerations with respect to the use of land equipment, in addition to the resistance requirements for the full horizontal water pressure. The necessity of a top berm and its size for sand dikes in the construction of quay walls cannot be determined by analogy with earth dams since, in this case, there is no need for imperviousness. There is also no need for construction roads, when floating equipment is used, and the horizontal pressure is also smaller than in the case of dams. Consequently

a small top berm has been used by the Bureau for all dikes in the construction of quay walls.

The successful use of this construction method did not extend to hydraulic clay fills; it did not establish the minimum dimensions of the dike required to be fully effective in reducing the horizontal pressure of the hydraulically placed fluid fill; nor did the installations lead to any clues by which to judge how, and in what manner, these reductions in the fluid pressure are effected.

Reduction in pressure can be conceived as resulting from the three following causes:

a. The structural strength or shearing resistance of the sand dike;
b. The continuous drainage course afforded by the sand dike; or
c. The increased friction of the sand against the sheet piles as compared to the friction of the fluid clay.

Therefore it was important to determine by test:

Case 1. The minimum dimensions of the sand dike to make it fully effective against an hydraulic clay fill (the sand dike may be of rectangular, triangular, or trapezoidal shape); and

Case 2. The variation of the effectiveness of the dike, with its size and dimensions, which would also furnish an indication as to how the reduction in the fluid pressure is accomplished.

The full effectiveness (case 1) of a sand dike is said to be realized, when the resultant pressure against the wall is the same as would have obtained had the entire fill behind the wall been made of the dike material. These were the two original objectives of the Princeton tests. The conclusions reached, based on the tests, are given in Professor Tschebotarioff's paper. One states that the full effectiveness of a sand dike is attained either when the dike is placed on the natural sand slope (1:1.73) or when a uniform vertical sand blanket of a thickness equal to height of the wall is interposed between the sheet pile wall and the fluid backfill.

The sloping dike was provided with a 1-ft berm on the top. In practice the sloping dike fill is the one to be used, since it can be placed more economically and also since the quantity of the backfill is less than the quantity required in the vertical blanket as long as the berm width is less than 0.135 times the height of the wall. Proposed typical design and construction procedures are shown in Fig. 38 for the two site conditions discussed at the beginning of this paper.

Another conclusion from the tests as given by Professor Tschebotarioff is that, if a vertical sand blanket of a thickness equal to one half of the bulkhead height is used, its effectiveness in decreasing the fluid clay pressure against the wall is reduced by one half and the effectiveness of the sand blanket is zero when its thickness is one tenth of the bulkhead height. The manner in which the effectiveness of the dike varies with the nature of the dike material —for instance, when gravel is used in the dike—was not determined. The present tests do not furnish enough data to find the variation of the effective-

ness of a vertical (rectangular shaped) sand blanket with its thickness. It is expected that the future tests will supply the missing data.

There are sufficient data, however, to indicate that the drainage has very little influence on the reduction of the fluid pressure against the wall. The

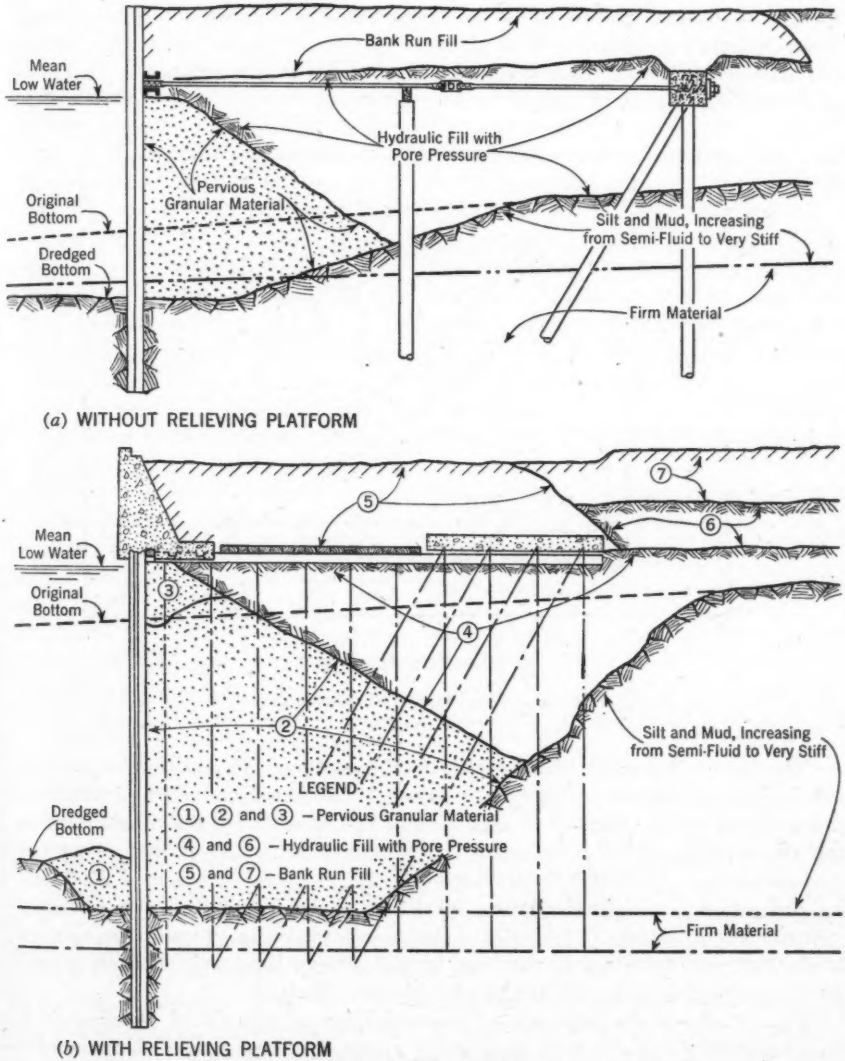
Bank Run Fill
Mean Low Water
Hydraulic Fill with Pore Pressure
Pervious Granular Material
Original Bottom
Dredged Bottom
Silt and Mud, Increasing from Semi-Fluid to Very Stiff
Firm Material
(a) WITHOUT RELIEVING PLATFORM
Mean Low Water
Original Bottom
Dredged Bottom
Silt and Mud, Increasing from Semi-Fluid to Very Stiff
LEGEND
① ② and ③ – Pervious Granular Material
④ and ⑥ – Hydraulic Fill with Pore Pressure
⑤ and ⑦ – Bank Run Fill
Firm Material
(b) WITH RELIEVING PLATFORM

FIG. 38.—PROPOSED METHOD OF BUILDING QUAY WALL

drainage effect is evidently the same whether the thickness is 0.1 or 0.5 of the wall height H; and yet a thickness of 0.1 H was entirely ineffective in reducing the fluid pressure of the clay fill. Although there can be no doubt that the

drainage factor must be operative, its effect is evidently too slow to make itself felt under the driving force of the material's own weight.

The action of the drainage course under a surcharge loading is not known. The influence of surcharge on the dimensions and the effectiveness of the sand dike is very important in the design of quay walls. No tests have been performed to date (October, 1947) to determine this influence. It is strongly suspected that a definite width of berm at the top of the sand dike will be required in this case for full effectiveness. This is one of the problems for which a solution will be sought in the continuing tests.

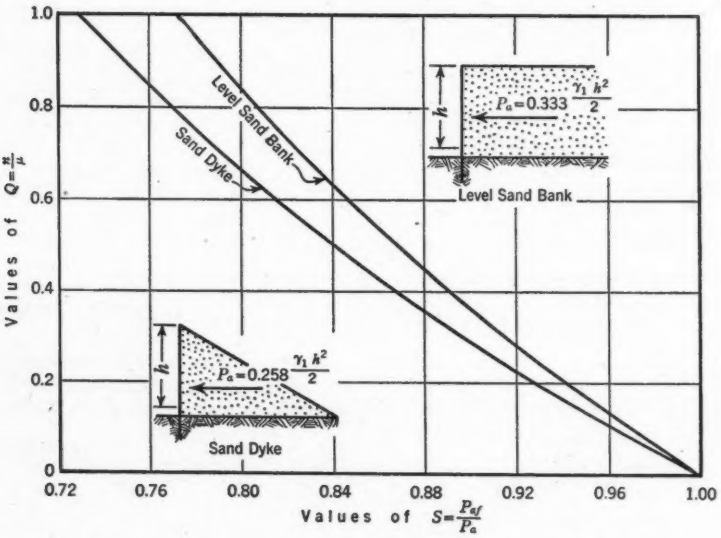

FIG. 39.—INFLUENCE OF WALL FRICTION ON ACTIVE SAND PRESSURES

The factor of wall friction, on the other hand, may play a more significant role in making the dike an effective barrier. The beneficial effect of wall friction is based on the transfer of a portion of the weight of the sliding wedge to the sheet piling, thereby reducing the weight that causes sliding along the rupture plane. Undoubtedly the interposition of a sand blanket of thickness $0.1\,H$ induces a potential friction force at the wall, and this was actually confirmed by the tests. The reason that no reduction in the pressure applied to the wall was recorded in this case is that a sand thickness of $0.1\,H$ is evidently not sufficient to render the wall friction effective.

Fig. 39 shows the influence of wall friction on the active pressure of a sand backfill, both for a level bank and a triangular dike. In Fig. 39, n denotes the tangent of the angle of wall friction; μ denotes the tangent of the angle of internal friction of the backfill material (0.577 for sand); γ_1 denotes the unit weight of sand; P_{af} denotes the maximum horizontal pressure per linear foot of wall at any height, h, and any value of n greater than zero; P_a denotes the

corresponding value of the horizontal pressure for $n = 0$; Q denotes $\frac{n}{\mu}$; and S denotes $\frac{P_{af}}{P_a}$. The capacity of any vertical section of the sliding wedge to transmit a portion of the weight of the wedge to the next adjoining section is measured by the shearing resistance of the section, namely $P_{af}\,\mu$. In no case will it be possible to mobilize an effective friction force, between the sand and the sheet piling, of magnitude greater than the shearing resistance of the backfill material adjacent to the wall. Since the available shearing resistance along a vertical section at the boundary between the sand dike and the fluid clay is zero, it follows that the reduction in the force that causes sliding can be made effective only for that portion of the wedge lying between the wall and the sand boundary—that is, for the portion of the wedge composed of sand.

Thus the maximum possible weight which can be transferred to the piling by wall friction is realized for the case where the entire sliding wedge is composed of sand—that is, for a vertical sand dike of a thickness equal to the width of the sliding wedge, or 0.577 H. However, even with this width of dike, the maximum possible reduction in the active pressure against the wall is comparatively small as can be seen from Fig. 39. The values of S for $Q = 1.0$ (that is, when the wall friction is equal to the internal friction of the sand) are 0.77 for level bank and 0.73 for a triangular dike, thus indicating values of 23% and 27%, respectively, for the maximum reduction in horizontal pressure resulting from wall friction. A reduction of this magnitude cannot account for the full effectiveness of a sand dike; the principal source of the dike's capacity to relieve the sheet piling must come from the third factor—namely, its structural strength.

The Conception of Structural Strength of the Triangular Sand Dike

Line AB, Fig. 40, is the original pressure against the wall exerted by the sand dike itself. The intensity of pressure at any distance h from the top is

$$p_s = \overline{ab} = K_1 \gamma_1 h \ldots\ldots (11a)$$

in which γ_1 is the buoyed weight of sand and K_1 is the active pressure coefficient for the triangular sand dike. Line AC, Fig. 40, is the pressure against the wall after the liquid fill is pumped in behind the sand dike. The intensity of pressure now, by reason of the full effectiveness of the dike, is

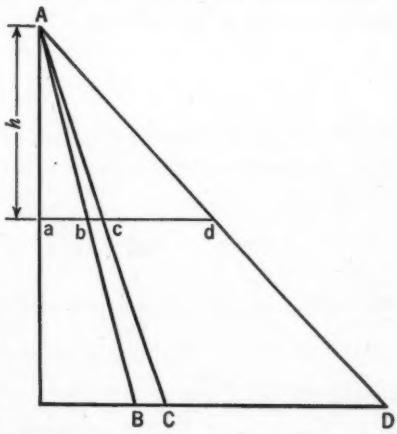

Fig. 40.—Comparison of Pressure Intensities

$$p_a = \overline{ac} = K_2 \gamma_1 h \ldots\ldots\ldots\ldots\ldots\ldots (11b)$$

in which K_2 is the active pressure coefficient for a level sandbank. The increase

in the intensity is

$$p = p_a - p_3 = \overline{ac} - \overline{ab} = \overline{bc} = \gamma_1 h (K_2 - K_1) \ldots\ldots\ldots (11c)$$

Line AD, Fig. 40, represents the pressure of the liquid clay against the sand dike, which is the same as for a wall with no interfering dike, with intensity of pressure at the same depth h of magnitude:

$$p_4 = \overline{ad} = \gamma_2 h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11d)$$

in which γ_2 is the buoyed weight of clay, the lateral pressure being the same as the vertical pressure. The difference between the pressure intensity against the wall and the fluid clay pressure intensity against the dike is

$$p_d = \overline{ad} - \overline{ac} = \overline{cd} = h (\gamma_2 - K_2 \gamma_1) \ldots\ldots\ldots\ldots (11e)$$

Thus, passing through the dike, the intensity of pressure dropped by the amount $\overline{cd}$, Fig. 40, because of the resistance of the dike, similar to a loss in potential across an electrical resistance, or reduction in head in a water pipe due to friction.

The resisting capacity of the dike may perhaps be better studied if, instead of intensities, total pressures are considered. The total active pressure, P_a, of a soil material with any positive or negative angle of surcharge, $\pm \theta$, and any angle of internal friction, ϕ, is

$$P_a = 0.5 K_a \gamma_e h^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

in which γ_e is the effective weight of soil and K_a is the active pressure coefficient determined from the relation:

$$K_a = \frac{\alpha_a - \mu}{(\alpha_a - \beta)(1 + \alpha_a \mu)} \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

In Eq. 13, α_a is the tangent of the angle X defining the plane of rupture and β is the tangent of the angle θ defining the surcharge. The tangent α_a is defined as

$$\alpha_a = \sqrt{(1 + \mu^2)\left(1 - \frac{\beta}{\mu}\right)} + \mu \ldots\ldots\ldots\ldots\ldots (14)$$

Similarly, the passive pressure coefficient is

$$K_p = \frac{\alpha_p + \mu}{(\alpha_p - \beta)(1 - \alpha_p \mu)} \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in which

$$\alpha_p = \sqrt{(1 + \mu^2)\left(1 + \frac{\beta}{\mu}\right)} - \mu \ldots\ldots\ldots\ldots\ldots (16)$$

The solution of Eq. 13 for representative values of $\mu = \tan \phi$ is presented in Fig. 41 and the solution of Eq. 15 is presented in Fig. 42.

In the sand dike case $\theta = -\phi = -30°$; therefore, $\frac{\beta}{\mu} = \frac{\tan \theta}{\tan \phi} = -1.0$ and K_a from Fig. 41 equals 0.258. Substituting for K_a in Eq. 12 gives the

total active pressure at height h. This procedure presupposes that the full shearing resistance of the sand dike has been mobilized. Eqs. 13 and 15 are ordinarily used in design as a standard procedure by the profession at large. The shearing strength of the sand dike is increased after the placing of the liquid

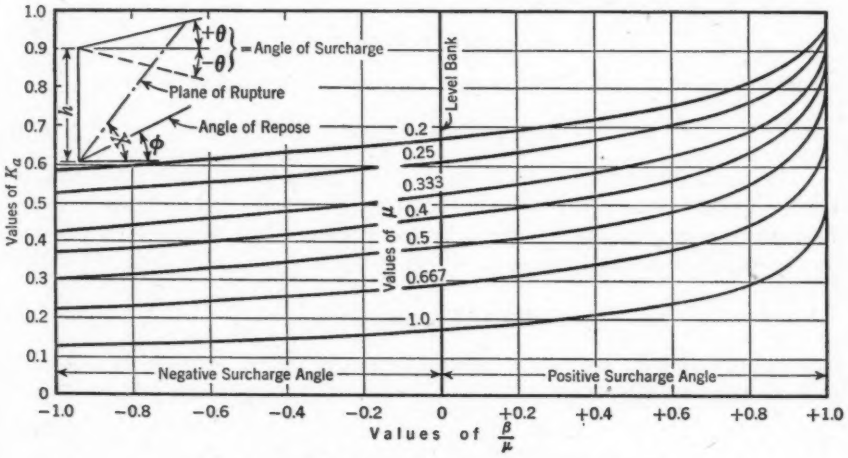

FIG. 41.—VALUES OF ACTIVE PRESSURE COEFFICIENTS K_a

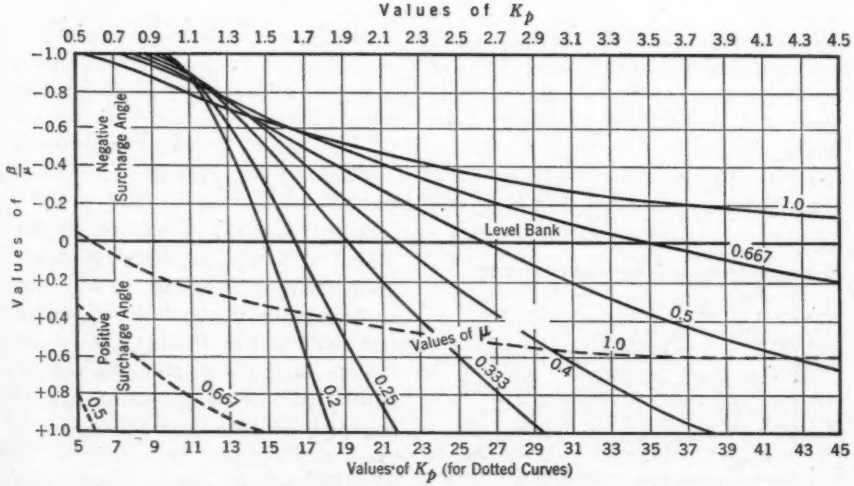

FIG. 42.—VALUES OF PASSIVE PRESSURE COEFFICIENTS K_p

clay by the increased normal pressure on the sloping dike surface produced by the buoyed weight of the clay fill; but the gain is small compared with the additional lateral pressure exerted by the liquid clay.

Consider the equilibrium of the trapezoidal prism ABCD, shown in Fig. 43, above an assumed rupture plane BC making an angle X with the horizontal.

The weight of the upper triangle ABC of the sand dike is

$$W_1 = \gamma_1 h^2/2 \, (\alpha_x + \mu) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

in which α_x denotes the tangent of angle X. The weight of triangle ACD of the clay fill on the rupture plane is

$$W_2 = \gamma_2 h^2 \mu/2 \, (\alpha_x + \mu)^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

The horizontal force of the clay fill on side CD of triangle ACD is

$$P_3 = W_2 \mu \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

The tendency of the prism to move down along the rupture plane toward the sheet pile wall results from the body forces W_1 and W_2 and from the force P_3. Resistance to movement is offered by the friction along the surface BC and by the wall's horizontal reaction, which is equal in magnitude to the active pressure

$$P_{ax} = P_1 + P_2 + P_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

in which P_1 and P_2 are the horizontal pressures due to weights W_1 and W_2 and are given by formulas:

$$P_1 = W_1 \, (\alpha_x - \mu)/(1 + \alpha_x \mu) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (21a)$$

and

$$P_2 = W_2 \, (\alpha_x - \mu)/(1 + \alpha_x \mu) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (21b)$$

The formulas are based on the assumption that the full shearing strength of the sand dike has been mobilized in each case. The active pressure (P_{ax}-curve) of the wall, at any depth h is plotted in Fig. 44 for various slopes of the rupture plane and for values of γ_1 and γ_2 of 60 lb per cu ft and 56 lb per cu ft, respectively. According to the conclusion obtained from the tests, the pressure should be the same as for a level sandbank, which, by Eq. 12, with $K_a = 0.333$ (Fig. 41 for $\mu = \tan 30°$) and $\gamma_e = 60$ lb per cu ft is $P_a = 0.5 \, K_a \, \gamma_e \, h^2 = 10 \, h^2$. The pressure is represented by the P_a-curve in Fig. 44, which is an arc of a circle with radius 10 h^2. For rupture planes in which the P_{ax}-curve falls within the P_a-curve, there is reserve strength in the wall and no additional shearing resistance is required in the sand dike. However, in the region from $X = 41°$ to $X = 68°$, the P_{ax}-curve falls outside the P_a-curve, showing that additional shearing resistance must be provided in the sand dike in order that the force against the wall should not exceed $P_a = 10 \, h^2$.

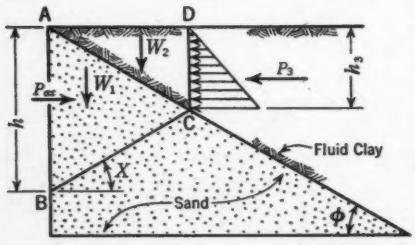

FIG. 43.—FORCES ACTING ON A TRIANGULAR DIKE

This strength may be provided in the form of a horizontal berm at the top of the sand dike. The inset diagram in Fig. 45 shows such a dike having a berm of width $t \, H$, t being a numerical factor less than 1.0. The weight of

quadrilateral ABCD of the sand dike above the rupture plane BC is

$$W_1 = \gamma_1 [1 + \mu t (2 + t \alpha_x) H^2/2 (\alpha_x + \mu) \quad \ldots\ldots (22)$$

The weight of triangle CDE of the clay fill on the rupture plane is

$$W_2 = \mu \gamma_2 (1 - t \alpha_x)^2 H^2/2 (\alpha_x + \mu)^2 \quad \ldots\ldots (23)$$

The horizontal force of the fluid clay on triangle CDE is

$$P_3 = W_2 \mu \quad \ldots\ldots (24)$$

The tendency of the prism to move downward along the rupture plane toward the wall results from the forces W_1, W_2, and P_3. Resistance to move-

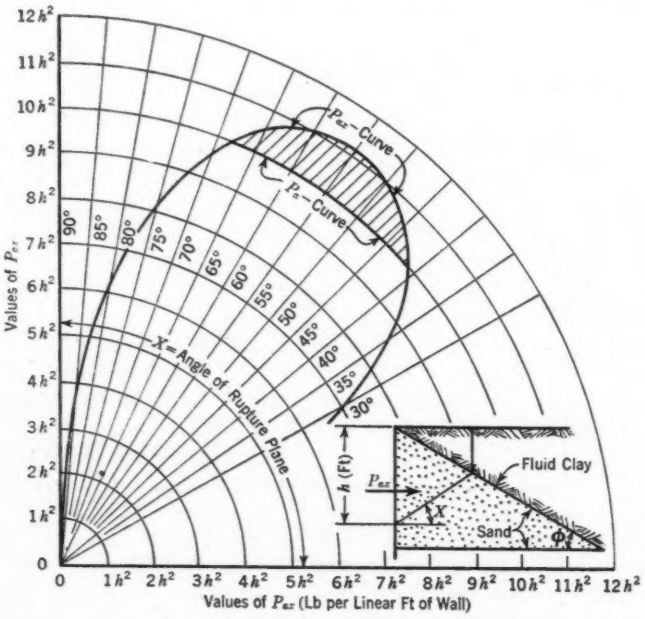

FIG. 44.—ACTIVE PRESSURES FOR TRIANGULAR DIKES

ment is offered by the friction along the surface BC and by the horizontal reaction P_{ax} of the wall which is equal in magnitude to the active pressure on the wall and is given by the formula:

$$P_{ax} = P_1 + P_2 + P_3 \quad \ldots\ldots (25)$$

in which P_1 and P_2 are the horizontal pressures due to weights W_1 and W_2 and are given by formulas:

$$P_1 = W_1 (\alpha_x - \mu)/(1 + \alpha_x \mu) \quad \ldots\ldots (26a)$$

and

$$P_2 = W_2 (\alpha_x - \mu)/(1 + \alpha_x \mu) \quad \ldots\ldots (26b)$$

The active pressures (P_{ax}-curve) at the total depth H is plotted in Fig. 45 for various slopes of the rupture plane and for three different values of $t = 0.1$, $t = 0.2$, and $t = 0.4$ and for values of $\gamma_1 = 60$ lb per cu ft and $\gamma_2 = 56$ lb per cu ft. The curve $P_a = 10\ H^2$ corresponding to the total pressure of a level bank is also plotted in Fig. 45. An analysis of these curves shows that the only P_{ax}-curve that falls entirely with the P_a-curve is the curve for $t = 0.4$—indicating that a sand dike to be fully effective must have a berm on top equal to 0.4 of the height of the wall. The berm provided in the test is considerably smaller. The discrepancy may be explained by the probable error in measurement readings, estimated to be approximately 5%. Errors of this magnitude

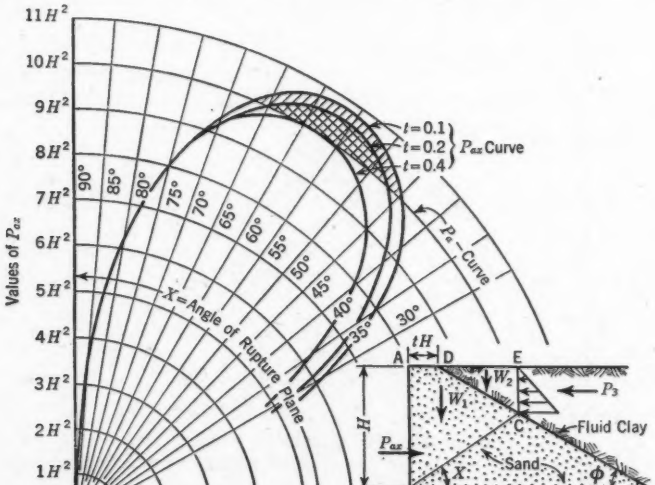

FIG. 45.—ACTIVE PRESSURES FOR TRAPEZOIDAL DIKES

are sufficiently large to account for the difference. Thus the maximum total pressure for the dike with berm width $t = 0.2$ is $10.5\ H^2$, whereas that for a level sandbank is $10.0\ H^2$—a 5% difference. Another possible explanation is that the fluid clay in the lower regions may have consolidated partly and did not exert full liquid pressure against the dike.

As the tests progressed, they branched out in various directions, and the scope of the objectives was extended to include studies of such problems as the determination of the K-value for liquid clay, the presence of arching in cohesive materials, and other related questions.

VERTICAL ARCHING

The Bureau of Yards and Docks has never taken advantage of arching in design, for the reason that arching is not considered a permanent phenomenon,

although the presence of arching in granular material has never been questioned. The arching problem was added to the objectives of this study, for the purpose of determining whether arching is also effective in cohesive material. The findings of Professor Tschebotarioff relative to this question, therefore, were in the nature of a big surprise. According to these findings, there is no arching even in granular material when the backfill is built up against the wall in steps. However, after a little thought, the absence of arching in this case seems to be reasonable, since the layers of sand during deposit are not restrained on the surface at each filling stage and the condition is somewhat similar to the cantilever wall where no arching is possible. Equally reasonable is the conclusion that the vertical arching, where present, is of an unstable character, resulting from the effect of compaction and vibration. This is the main reason why the Bureau always neglected the vertical arching effect in all designs of flexible quay walls.

Design Considerations and Recommendations

The fact that the stability of a bulkhead depends on two components of different materials—soil and the structural members—raises an economical consideration that is of even greater importance to the structural engineer than it is to soil scientists. It is a known fact that there are no quay wall failures on record that can be attributed to failures of sheet piles; most of those recorded are the result of the movement of the upper anchors and, to a limited extent, of insufficient penetration of piles at the bottom. Thus, the failures of bulkheads are largely the fault of the soil and not the structural members. In other words, sheet piles do not fail in bending and tie rods do not fail in tension. The reason for this can be mainly ascribed to the fact that the structural members are designed for a much higher factor of safety than that usually allowed for the soil. In any design, if the full values for the angle of internal friction, cohesion, and wall friction (as found in the laboratory from boring samples) are used—the usual and accepted procedure—a factor of safety is entirely lacking in the bulkhead in so far as the soil resistance is concerned.

It has been customary to neglect the cohesion factor for all soils having an angle of internal friction greater than about 17° in computing both the active and passive pressures according to the formulas (for active pressures):

$$P_a = \gamma_s h \tan^2\left(45° - \frac{\phi}{2}\right) - 2c\tan\left(45° - \frac{\phi}{2}\right) \quad \ldots\ldots\ldots (27a)$$

and (for passive pressures)

$$P_p = \gamma_s \tan^2\left(45° + \frac{\phi}{2}\right) + 2c\tan\left(45° + \frac{\phi}{2}\right) \quad \ldots\ldots\ldots (27b)$$

in which c is the cohesion of the soil per unit area. This procedure has been followed purposely to provide the missing factor of safety for the soil. This factor of safety may be only imaginary, however, and cannot be depended on. In the first place, friction and cohesion may not act simultaneously, the resistance of the two being somewhat similar to the resistance offered by a joint partly welded and partly riveted. The frictional resistance of the soil is

mobilized almost at once whereas the mobilization of the cohesive resistance requires appreciable movement, and it is doubtful whether the deflection of the bulkhead allows enough movement in the soil to mobilize the full cohesion. The omission of any beneficial effect from arching, even in cases where arching exists and could be depended on, affords a greater factor of safety for sheet piles; but it does not help the soil situation. The only reliable factor of safety realized in conventional designs is the omission of the beneficial effect of wall friction in computing the active and passive pressures. However, the magnitude of the factor of safety afforded by this omission is not considerable.

Fig. 39 shows the variation of the active pressure with the angle of wall friction for a sand fill for either a level bank or a dike. As noted from the curve, the maximum possible reduction in the active pressure, when the angle of the wall friction is equal to the angle of internal friction, is only about 23% for a level bank, which is equivalent to a factor of safety of 1.3. The omission of the wall friction for the passive pressure will probably raise the factor of safety to about 1.75. When computing the resistance of the wall to a Swedish slide, most designers consider themselves lucky when they obtain a factor of safety of 1.50. Moreover, the factor of safety of steel is based on the elastic limit which ordinarily means a factor of safety of about 4, on the ultimate strength, whereas the factor of safety of the soil will seldom, if ever, reach a value of 2.0. The disparity between the factors of safety of the two materials will not be affected by any increase in the active, or reduction in the passive, pressures, since the sizes of the structural members will be increased by this procedure in the same proportion. Since the strength of a chain is determined by its weakest link, there is evidently an undue waste of money and material resulting from the use of present design methods. A recommendation to increase the allowable unit stress in the structural parts to approximately the elastic limit of the material would seem to be in order. As to the factor of safety of the soil, it would seem logical to adopt the following procedure: For the active pressure, (*a*) use the full values for the angle of internal friction, cohesion, and wall friction as found from tests, (*b*) apply a factor of safety of 1.5 to the theoretical values of the passive pressures in determining the size and dimensions of the upper anchor reactions, and (*c*) omit the effect of wall friction in figuring passive pressures.

Model Test Requirements

To derive more or less rational formulas from model tests, the model must include all the variables that may have an influence on the problem to be solved. Then, and only then, can a rational interpretation of the model results be made and a proper application to the prototype be assured. The problem of pressure distribution on flexible walls involves the interaction of two different materials—the soil and the sheet piles. It should be studied, therefore, not only from the points of view of the elasticity and the plasticity of the soil, but also from the points of view of the elasticity and the stiffness of the sheet piling. The magnitude and the distribution of pressures on the bulkhead must be such that the corresponding deformations of the soil, obeying its elastic and plastic laws, must coincide with the elastic deflection curve produced

by these pressures in the sheet piles according to the laws governing the piles, their modulus of elasticity, and their stiffness—that is, the magnitude and the distribution of pressures is controlled by the boundary condition between the soil and the sheet piles.

The problem is similar to a foundation on an elastic medium. The Boussinesq law of pressure distribution is not applicable when the load is transferred to the soil through foreign mediums such as concrete slabs and beams until the contact pressure distribution between slab and soil has been determined. In such cases, the moment of inertia and the modulus of elasticity of the transferring medium, as well as the modulus of the soil reaction, determine the distribution of the soil pressures. The variation in the wall stiffness may not be as important in the determination of the total pressure on the wall; it is of great importance, however, in the determination of pressure distribution, which seems to baffle all experimenters. It would seem that, in the future, model tests should include the wall stiffness variable in the hope that the results may lead to a rational pressure distribution formula. It is felt that the tests reported in this Symposium have advanced the knowledge of the behavior of flexible bulkheads considerably, but much remains to be learned. It is hoped that the exceptional facilities of this model, and the new improved testing methods used, will encourage and spur on further research in the indicated direction. Such a study will require considerable effort both in resources and patient work, but the task is well worth doing.

EXPERIENCES WITH SOIL TYPES IN NAVAL CONSTRUCTION

By L. A. Palmer,[38] Esq.

Synopsis

The principles of soil mechanics are demonstrated by two examples in this paper. One concerns pile foundations in plastic clay, and the other concerns the stability of an extensive mass of sand deposited as a foundation material for a cellular pier, comprised of two rows of cells of interlocking sheet piling. A rather unusual condition in an homogeneous clay deposit, 90 ft or more deep, of apparently sedimentary origin is described. The density and moisture content of the clay are apparently unrelated as to depth. Consolidation and shear curves are shown. Two expressions are derived, one for the case of capillary saturation and the other for the case of complete submergence, which show that the in-place consolidating pressure of fully consolidated soil (homogeneous as to type and without structural load) is an exponential function of depth.

A critical density study of the sand used both to fill and to support the steel cells of the cellular piers is described. The purpose of this study was to indicate the possible decrease in stability of the sand mass that occurs as the result of severe earthquake shock.

Introduction

During the decade before World War II, there were many advances on a broad front in the field of theoretical and applied soil mechanics. Prior to this time a splendid groundwork had already been laid by Professor Terzaghi and his associates and the many researches that later followed this pioneering made a wealth of valuable material available for utilization in the large wartime construction program of the United States Navy.

During this time the Bureau of Yards and Docks, like the Corps of Engineers of the United States Army, was early confronted with the job of converting poor foundation soils into good soils, a procedure known as "soil control." This procedure, once frequently referred to as "soil stabilization" in highway construction, had its place in the subgrade preparation for paving Naval and Marine Corps air stations, in the stiffening and consolidation of mud deposits by vertical sand drains, in the compaction of sand by vibroflotation, and in the substitution of sand dikes for mud as backfill material.

This paper is concerned with a few of the physical properties of two very distinct types of soils, plastic clay and sand, as they were encountered on two

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Naval construction projects. Both projects involved principles of soil mechanics that are typical of those that have been receiving considerable study.

Plastic Clay

In connection with the design of two rigid frame hangars for blimps at Houma, La., the foundation soil exploration included deep borings, soil sampling and testing, plate loading tests in open pits from 7 ft to 9 ft deep, and static load tests on timber piles. The low bearing values obtained by plate loading tests precluded any notion of carrying the A-frame loads on spread footings. Beginning at a depth of from 10 ft to 20 ft below the ground surface and extending to a total depth of 90 ft or more, the soil was composed of a mixture of silt and clay. At least 90% of all samples could be washed through the No. 270 sieve. The clay fraction contained 20% or more of colloids. The shapes of the consolidation curves indicated no definite relationship between the depth of overburden and the preconsolidation pressure (vertical intergranular pressure) of the clay soil in place. The bulk densities of samples from one of the borings, taken at depth intervals of 10 ft, did not vary more than 2 lb per cu ft. The unit cohesion of the clay obtained by unconfined compression tests was somewhat variable, ranging from 450 lb per sq ft to 750 lb per sq ft, independently of the depth. The computed unit skin friction of the test piles varied from 500 lb per sq ft to 750 lb per sq ft, on the average being somewhat more than

TABLE 1.—Observations on Test Piles at Hangar A, United States Naval Air Station, Houma, La. (Hammer No. IV)

Pile No.	Diameter (In.)		Blows per ft[a]	Penetration (ft)	Contact area (sq ft)	Settlement, in Feet, at:		Total load (tons)	Unit Skin Friction (Lb/Ft²)		Rebound (ft)[c]
	Butt	Tip				30 tons	Maximum load		Observed	Computed[b]	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1–60	15	10	6¼	51	175.0	0.007	0.027	57.2	654	571	....
1–80	20	10	11¾	71	256.0	0.007	0.029	60.0	....[d]	....[d]	0.016
2–80	20	10	11¾	....	256.0	....	0.039	90.0	704	486	....
2–60	18	10	8	51	186.0	0.008	0.028	66.45	714	552	....
2–80	17	9	7	71	236.0	0.011	0.035	80.0	680	506	....
5–60	15	10	7½	51	167.0	0.007	0.018	60.0	721	600	....
5–80	19	11	15½	71	280.0	0.012	0.036	75.0	535	424	0.017
6–60	15	8	5	51	149.2	0.008	0.017	55.4	740	550	....
6–80	21	9	16	71	254.5	0.011	0.038	89.1	700	457	....

[a] Determined by computing the average of the blows in the last 3 ft of penetration. [b] The values of skin friction (pounds per square foot) in Col. 11 are as computed, for piles of constant diameters equal to the butt diameters, for the entire lengths. The average of these values is about the same as the average unit cohesion of the clay. By this method, average skin friction is equal to 518 lb per sq ft, whereas the average cohesion is equal to 535 lb per sq ft. [c] Except as indicated for pile Nos. 1–80 and 5–80, each pile was driven to failure. For the piles that failed, the computed average pile friction was 701 lb per sq ft (see Col. 10). The rebound of pile No. 6–80 could not be determined. [d] The load was removed after loading to 60 tons. Upon reloading, this pile failed under a load of 90 tons.

the average unit cohesion of the clay. If it is assumed that the clay sheared to form, with the pile, a cylinder of diameter equal to that of the pile butt, then the average computed skin friction is about the same as the average unit cohesion of the clay.

Table 1 contains pile load test data at the site of one of the two hangars. Fig. 46 shows plotted triaxial test data. No significant effective angle of internal friction ϕ was evident from shear tests made by either the direct method (quick shear) or the triaxial device in which the tests were also made quickly and without loss of moisture during the test.

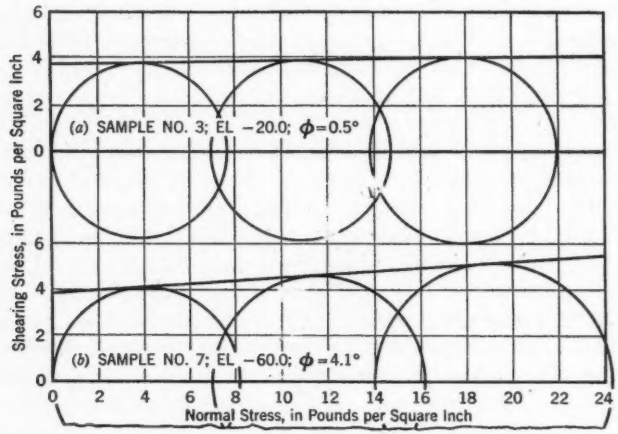


FIG. 46.—TRIAXIAL SHEAR TEST DATA; PLASTIC CLAY; HANGAR NO. 2, HOUMA, LA.; $c = 5.0$ LB PER SQ FT

It was concluded that settlement by slow consolidation of the clay, under the load of the piles, would not be realized for the reason that there could be little possibility of relief of pore pressure developed by axial loads on piles. The few small local sand lenses at considerable depths below the estimated required depth of penetration of the piles, could not act as drainage courses for egress of pore water. Furthermore, the computed pore pressure delivered to surrounding soil was practically zero for the top several feet below the ground surface so that there was no driving force to carry the flow of water entirely to the surface. No appreciable settlements were reported during the 4 years following construction.

The moisture content of this clay varied from 30% to 75% (dry weight basis) with most of the values ranging from 35% to 50%. The moisture content was entirely independent of the depth.

This clay belongs in the category of those described by Karl Terzaghi,[18] M. ASCE, Ralph B. Peck,[17] Assoc. M. ASCE, and W. S. Housel,[39] M. ASCE, with one notable exception—in the laboratory shearing tests the measured angle of internal friction in the present case did not exceed 5° whereas the laboratory value reported by Professor Peck was $\phi = 17°$.

The possibility of buckling of timber piles driven to a total penetration of 80 ft, the lower 2 ft being in sand lenses, might be considered for the reason that the clay tended to deform noticeably at stresses very much less than those

[39] "Earth Pressure on Tunnels," by W. S. Housel, in "Earth Pressure and Shearing Resistance to Plastic Clay: A Symposium," *Transactions*, ASCE, Vol. 108, 1943, pp. 1037–1058.

corresponding to failure. Thus, in plate loading tests at 9 ft or more below grade, a unit load of 1 ton per sq ft produced a settlement of 1 in. although the bearing value was theoretically at least $(\pi + 2)\,c$, or 3,100 lb per sq ft, with a unit cohesion (c) of 600 lb per sq ft which is characteristic of the clay under the loading plate. Using the expression:[40]

$$P_{cr} = \frac{\pi^2 E I}{L^2_p}\left(m^2 + \frac{D K L^4_p}{m^2 \pi^4 E I}\right) \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

in which the factor m can be evaluated by the expression:

$$m^2 (m + 1)^2 = \frac{D K L^4}{\pi^4 E I} \ldots\ldots\ldots\ldots\ldots\ldots (29)$$

Let the length of the pile $L_p = 80$ ft; $E = 216{,}000{,}000$ lb per sq ft; $I = \pi/64$; the average diameter of the pile, $D = 1$ ft; and $K = 24{,}000$ lb per cu ft. Assuming that the factor m is 5 (Eq. 29 solved to the nearest whole number), Eq. 28 yields $P_{cr} = 1{,}031{,}000$ lb. This value is in excess of 500 tons, which is three times the compressive strength of the cypress piles.

The Relation of Pressure to Depth in Natural Soil Deposits

The equation of the straight line part of a typical consolidation curve, with e, the void ratio, plotted against $\log_{10} p_s$, the consolidation pressure, is

$$e = B - A' \log_{10} p_s \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which A' is the slope of the semilogarithmic plot of the curve; and B is the intercept of the straight line part of the curve at the ordinate 1 ton per sq ft.

Converting $\log_{10} p_s$ to $\log_e p_s$, Eq. 30 becomes

$$e = B - \frac{1}{2.3026} A' \log_e p_s \ldots\ldots\ldots\ldots\ldots\ldots (31a)$$

or

$$e = B - A \log_e p_s \ldots\ldots\ldots\ldots\ldots\ldots (31b)$$

in which $A = \dfrac{1}{2.3026} A'$ and e is the Naperian base of logarithms.

Now consider a homogeneous clay type soil of indefinite depth and extent and assume the following conditions:

1. The ground water level exists at a depth H_g below the surface;
2. Water has risen by capillarity to the ground surface and all voids are filled with water;
3. The constants, A and B, are characteristic of the entire soil mass; and
4. The soil is completely consolidated by the in-place vertical load at all depths.

The soil at the surface is subjected to capillary pressure, p_c,

$$p_c = \gamma_w H_g \text{ (at ground surface)} \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

γ_w being the unit weight of water.

[40] "The Stability of Foundation Piles Against Buckling Under Axial Load," by A. E. Cummings, *Proceedings*, Highway Research Board, National Research Council, Washington, D. C., Vol. 18, 1938, Pt. II, pp. 112–119.

At any depth, h_o, less than H_g, the vertical intergranular pressure, or the consolidation pressure, p_s, is

$$p_s = \gamma_w (H_g - h_o) + \gamma_a h_o \quad \ldots\ldots (33)$$

γ_a being the unit weight in air of the saturated soil. Then at the depth $h_o + dh_o$, the consolidation pressure is $p_s + dp_s$. The average γ_a-value of the soil within dh_o is $\frac{\gamma_a + (\gamma_a + d\gamma_a)}{2}$ and

$$dp_s = \left(\gamma_a + \frac{d\gamma_a}{2}\right) dh_o = \gamma_a\, dh_o + \tfrac{1}{2}\, d\gamma_a\, dh_o = \gamma_a\, dh_o \quad \ldots\ldots (34)$$

Rewriting Eq. 31*b*,

$$p_s = e^{(B-e)/A} \quad \ldots\ldots (35)$$

from which

$$dp_s = -\frac{e^{(B-e)/A}}{A}\, de \quad \ldots\ldots (36)$$

by substitution of Eq. 36 in Eq. 34,

$$\gamma_a\, dh_o = -\frac{e^{(B-e)/A}}{A}\, de \quad \ldots\ldots (37)$$

also

$$\gamma_a = \frac{G + e}{1 + e}\gamma_w \quad \ldots\ldots (38)$$

G being the specific gravity of the soil solids. From Eq. 33,

$$p_s = \gamma_w H_g + h_o (\gamma_a - \gamma_w) = \gamma_w H_g + h_o \left(\frac{G + e}{1 + e} - 1\right) \gamma_w \quad \ldots (39a)$$

Therefore,

$$\gamma_a\, dh_o = \gamma_w \left(\frac{G - 1}{1 + e}\right) dh_o \quad \ldots\ldots (39b)$$

Substituting Eq. 39*b* in Eq. 37:

$$\gamma_w \left(\frac{G - 1}{1 + e}\right) dh_o = -\frac{e^{(B-e)/A}}{A}\, de \quad \ldots\ldots (40)$$

Eq. 40 is now integrated but not between limits. The constant of integration is

$$\text{Constant} = -\frac{e^{(B-e_s)/A}\,(1 + e_s + A)}{\gamma_w\,(G - 1)} \quad \ldots\ldots (41)$$

in which e_s is the void ratio at the surface, $h_o = 0$. That is,

$$e_s = B - A \log_e \gamma_w H_g \quad \ldots\ldots (42)$$

so that, finally,

$$h_o = \frac{p_s [1 + B + A\,(1 - \log_e p_s)] - \gamma_w H_g [1 + B + A\,(1 - \log_e \gamma_w H_g)]}{\gamma_w\,(G - 1)} \quad \ldots (43)$$

In Eq. 43, the constants, A and B, and the value, G, are laboratory values and H_g is known from boring data. If an arbitrary value is assigned to p_s,

the corresponding value for h is obtained from Eq. 43. Thus, the relation between depth and intergranular vertical pressure is exponential and not linear, as often assumed.

If the ground water is constantly at the surface, the consolidation pressure is zero at the surface. The consolidation pressure at any depth h below the ground surface is $(\gamma_s - \gamma_w)\,h$ in this case. The relation between h and e is again

$$h_o = \frac{e^{(B-e)/A}\,(1 + e + A)}{\gamma_w\,(G - 1)} + \text{constant} \qquad (44)$$

but the constant of integration in Eq. 44 differs from that obtained in the case of capillary saturation.

It may be considered that the soil is at its liquid limit at the ground surface. Since the condition of complete consolidation is assumed as existing at all depths, Eq. 31*a* should be applicable at all finite depths that correspond to values of p_s that are within the range of values, for this variable, included in the straight line part of the laboratory-determined consolidation curve. F. A. Robeson[41] has shown, however, that for certain conditions, the straight line part of the consolidation curve applies only for the values of p_s equal to or greater than 0.1 ton per sq ft. It may then be assumed that from $h = 0$ to a depth h_i where $(\gamma_s - \gamma_w)\,h_i = 0.1$ ton per sq ft, the soil mass is at its liquid

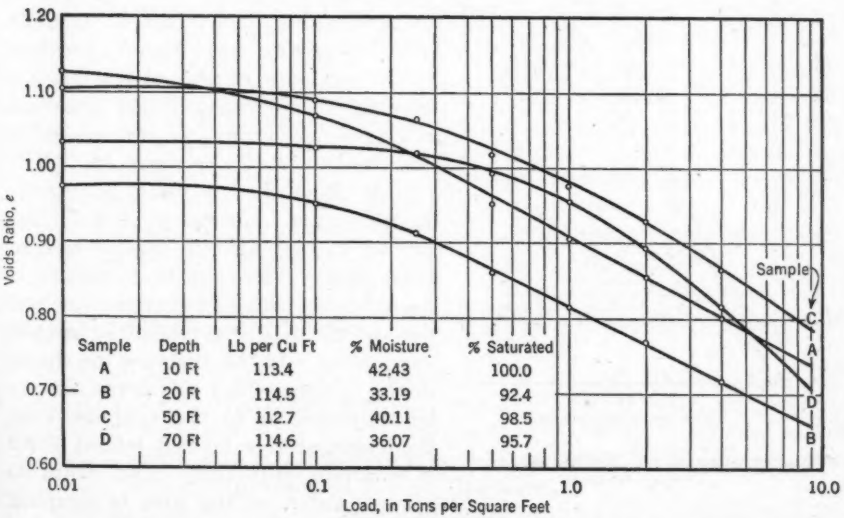


FIG. 47.—TYPICAL CONSOLIDATION TEST DATA, PLASTIC CLAY; HANGAR NO. 2, HOUMA, LA.

limit and that below h_i the linear relation between e and $\log_e p_s$ expressed by Eq. 31*a* obtains. Then, from Eq. 44,

$$\text{Constant} = h_i - \frac{e^{(B-e)/A}\,(1 + e_i + A)}{\gamma_w\,(G - 1)} \qquad (45)$$

[41] "A Method of Predicting Settlement of Fills Placed on Muck Beds," by F. A. Robeson, *Public Roads*, February, 1936, p. 249.

in which e_i is the void ratio corresponding to $p_s = 0.1$ ton per sq ft; and, hence,

$$h_o - h_i = \frac{e^{(B-e)/A}(1 + e + A) - e^{(B-e_i)/A}(1 + e_i + A)}{\gamma_w (G - 1)} \quad \ldots\ldots (46)$$

but, as before, $e^{(B-e)/A} = p_s$ and $1 + e + A = 1 + B + A(1 - \log_e p_s)$. Also, $e^{(B-e_i)/A} = 0.1$ ton per sq ft and $1 + e_i + A = 1 + B + A(1 - \log_e 0.1)$. Then, by substitution in Eq. 46,

$$h_o - h_i = \frac{p_s [1 + B + A(1 - \log_e p_s)] - 0.1 [1 + B + A(1 - \log_e 0.1)]}{\gamma_w (G - 1)} \quad \ldots (47)$$

Eq. 47 again shows (and this time for the case of complete submergence) that the change in consolidation pressure, p_s, with depth, in soil that is completely consolidated by the existing vertical pressure is an exponential relationship. It is obvious from consolidation curves that the consolidation pressure is an exponential function of the void ratio, but it is not obvious that p_s is also an exponential function of depth in a homogeneous soil type that is completely consolidated under weight of overburden plus any existing capillary pressure.

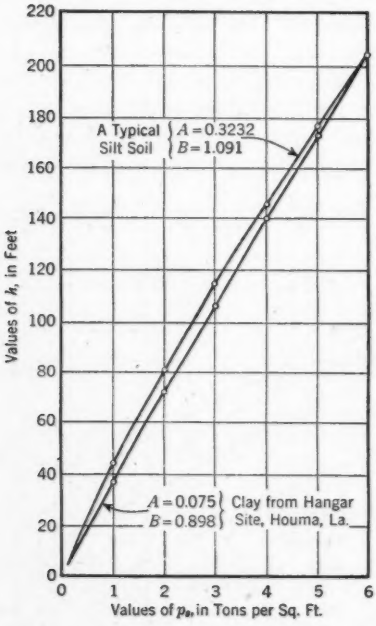


FIG. 48.—CURVES OF EQ. 34, FOR TWO TYPICAL SOILS

In the case of the plastic clay at Houma, the preconsolidation pressures, estimated according to the procedure described by A. Casagrande, M. ASCE, and R. E. Fadum,[42] Assoc. M. ASCE, varied from 0.5 kips per sq ft to 3.0 kips per sq ft, throughout depths ranging from 10 ft to 70 ft and there seemed to be no good relation between these values and the depths at which the samples were taken. In this instance the shapes of the (e versus $\log_e p_s$)-curves did not lend themselves to the analysis, since there seemed to be no well-defined points of sharpest curvature. The depth to ground water, at the time of sampling was 4 ft. The estimated preconsolidation pressures were generally somewhat less than the estimated overburden loads, indicating incomplete consolidation of the deposit.

Fig. 47 shows four typical consolidation curves for samples of this clay taken at depths of 10, 20, 50, and 70 ft. The bulk density and moisture content (dry weight basis) of the same samples, as received, are given in Table 2. Fig. 48

[42] "Notes on Soil Testing for Engineering Purposes," by A. Casagrande and R. E. Fadum, *Soil Mechanics Series No. 8*, Harvard Univ., Cambridge, Mass., January, 1940.

shows two graphs of Eq. 47 for two soils, one for the plastic clay at Houma, and the other for a typical silt soil from another source. The greater the increase in unit weight produced by progressively increasing consolidation pressures, the more the graphs of Eqs. 43 and 47 depart from a linear relationship between depth and consolidating pressure. For a constant unit weight, unchanging with depth, the relation would be strictly linear for a homogeneous soil. The graph for the plastic clay, Fig. 48, is almost linear. This fact, in itself, indicates small changes in density with depth even for a condition of complete consolidation of the clay in place.

TABLE 2.—TEST DATA, PLASTIC CLAY SAMPLES IN FIG. 47

Sample	Depth (ft)	Unit weight (lb per cu ft)	Moisture content (%)	Percentage saturation
A.....	10	113.4	42.43	100.0
B.....	20	114.5	33.19	92.4
C.....	50	112.7	40.11	98.5
D.....	70	114.6	36.07	95.7

SAND

Fig. 49 shows a typical section through a cellular pier, approximately 400 ft wide with berth lengths along the two sides of 1,700 ft and 1,000 ft, respectively. This project was on the west coast at Hunters Point, California. The periphery of the pier is bounded by a continuous series of steel sheet pile cells driven into a deep sand plug, deposited for this purpose. Those cells in which steel H-piles were driven for the crane tower foundations are not shown in Fig. 49. It was considered that the driving of the H-piles would tend to increase the density of the sand in and below these cells to such an extent that there would be no serious question as to sudden loss of stability by tectonic forces.

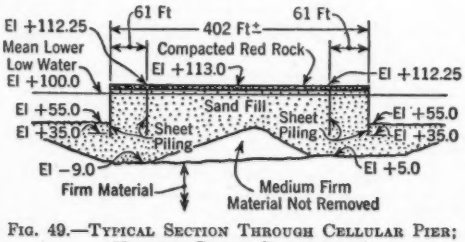


FIG. 49.—TYPICAL SECTION THROUGH CELLULAR PIER; HUNTERS POINT, CALIFORNIA

The sand was obtained from various sources on this project, yet it was surprisingly uniform in gradation. In the main, it was a poorly graded, medium sand, with from 30% to 60% passing the No. 40 sieve and retained on the No. 60 sieve, with practically no material retained on the No. 10 sieve and with very little material passing the No. 200 sieve.

Triaxial and direct shear tests were made with the sand under variable conditions with respect to void ratio and density of the sand. The direct and triaxial shear tests were made according to the procedures described by Professors Casagrande and Fadum.[42] Fig. 50 shows the relationship between the initial void ratio, e, the minor principal stress, p_2, maintained constant during an individual triaxial test, and the indicated percentage change in volume that accompanies a change from the initial to the critical void ratio. It may be noted from Fig. 50 that a definite critical void ratio, e_{cr}, corresponds to each

separate value of minor principal stress, p_2. This method of analysis has been treated in detail by D. W. Taylor,[43] and by Professor Casagrande and J. D. Watson.[44]

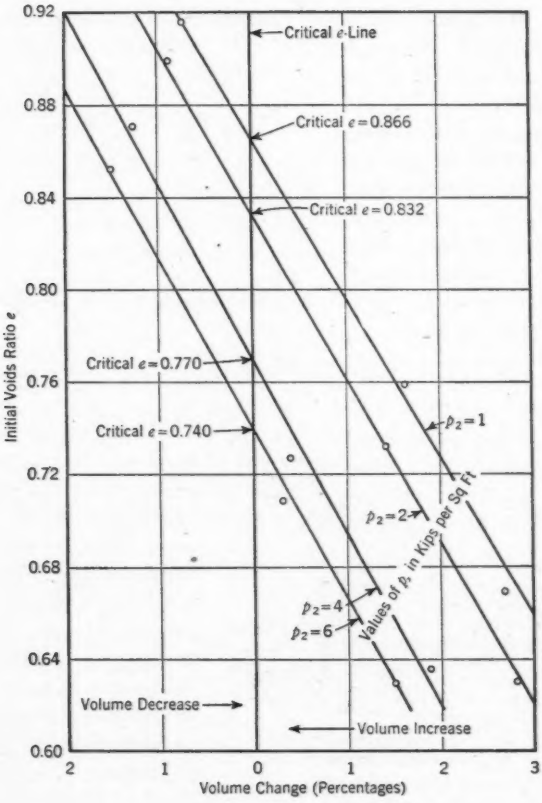


FIG. 50

Beginning with the well-known Rankine relation between the principal stresses p_1 (major) and p_2 (minor), Fig. 51 reveals that

$$v = p_n \tan \phi \quad \ldots\ldots (48)$$

Also,

$$p_n = \overline{OB} = \overline{OC} - \overline{BC} = \frac{p_1 + p_2}{2} - \frac{p_1 - p_2}{2} \sin \phi$$

$$= \frac{p_1 (1 - \sin \phi) + p_2 (1 + \sin \phi)}{2} \quad \ldots (49)$$

[43] "A Comparison of Direct Shear and Cylindrical Compression Tests," by Donald W. Taylor, in "Symposium on Shear Testing of Soils," *Proceedings*, ASTM, Vol. 39, 1939, p. 1058.

[44] "Compaction Tests and Critical Density Investigation of Cohesionless Materials for Franklin Falls Dam, Merrimack Flood Control," by A. Casagrande and J. D. Watson, Corps of Engrs., U. S. Army, Boston, Mass., April 1, 1938, Appendix BII, pp. 1–38.

and, since,

$$p_1 (1 - \sin \phi) = p_2 (1 + \sin \phi) \ldots\ldots\ldots\ldots\ldots\ldots (50)$$

it follows that

$$p_n = p_2 (1 + \sin \phi) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

Consequently,

$$v = \text{shearing resistance} = p_2 (1 + \sin \phi) \tan \phi \ldots\ldots\ldots\ldots (52)$$

It is apparent from Fig. 50 that with a relatively high minor principal stress the reduction in volume of the sand from its loose state to the critical density (at which it shears at constant volume) may be several times the volume reduction accompanying the transition from loose to critical state with a relatively low minor principal stress. The appreciable decrease in volume of the submerged sand requires considerable egress of water from the large mass of sand. Since no sand is permeable enough to render the escape of water instantaneous, a very sizable pore pressure would tend to be developed and maintained for a finite time if there were a sudden and severe earthquake. Under these conditions the combination of static overburden load and vibration would tend to effect a sudden diminution in volume of the sand mass, which in the lower reaches would trap the water and convert a considerable part of the intergranular stress to pore pressure. In the absence of any vibration, the sand mass, even in the loose state, is capable of developing the full shearing resistance as expressed in Eq. 52.

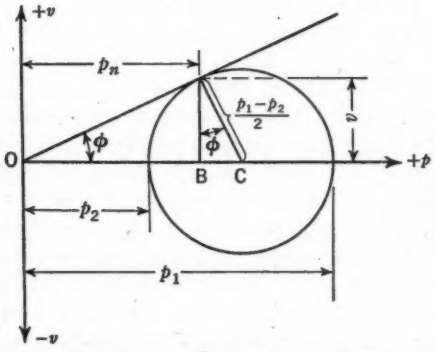


FIG. 51.—SHEARING RESISTANCE OF COHESIONLESS SOIL AT FAILURE

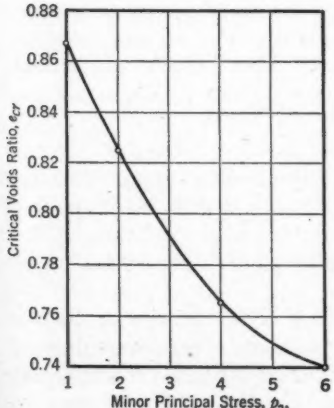


FIG. 52

Thus, at El. + 35 (Fig. 49) and in the mole back of the cells, if p_1 is taken as approximately equal to the weight of overburden (5.00 kips per sq ft); and, with ϕ equal to 28°, p_2 by Eq. 50 equals $\frac{0.53}{1.47} \times 5 = 1.80$ kips per sq ft and the shearing resistance v, by Eq. 52, is $1.8 \times 1.47 \times 0.532 = 1.41$ kips per sq ft.

In Fig. 52 the critical void ratios are plotted against the corresponding p_2-values of the triaxial tests. Using the data of Fig. 50, the critical void ratio for $p_2 = 1.8$ kips per sq ft is 0.834. If the void ratio at El. + 35 and between the rows of cells (see Fig. 49) does not exceed 0.834, there is an indicated mini-

mum shearing resistance of 1.41 kips per sq ft. However, if the void ratio should be at 0.866, or slightly above this value, then, from Fig. 50, the effective intergranular value of p_2 could not exceed 1 kip per sq ft; and, for this condition, Eq. 52 yields $v = 1 \times 1.47 \times 0.532 = 0.78$ kips per sq ft. Thus, a sudden disturbance would tend to reduce the shearing resistance to approximately one half of its value under conditions of static load. The intergranular value of p_1, with disturbance, would become, by Eq. 40, $\frac{1 + \sin 28°}{1 - \sin 28°} \times 1 = 2.78$ kips per sq ft. Consequently the temporary pore pressure developed would be $5.00 - 2.78 = 2.22$ kips per sq ft.

At the outboard side or toe of the cells, because of the rotating moment, p_1 is considerably greater than the weight of overburden (5 kips per sq ft) at the lower ends of the sheet piling. Without going into details of the computations of the vertical pressure distribution at this level within the cells, it may be stated that the computed maximum vertical pressure at the toe is 7.5 kips per sq ft. It is recognized that the major principal stress at this point very likely exceeds this value but a complete and satisfactory stress analysis, applicable to this region, is nonexistent to the best of the writer's knowledge. For an assumed or estimated value of 7.5 kips per sq ft for p_1, the corresponding value for p_2 by Rankine's relation would be 2.7 kips per sq ft, and the computed shearing resistance from Eq. 52 is 2.10 kips per sq ft. From Fig. 52, for $p_2 = 2.7$, the critical void ratio is 0.80; and, within this region, with e equal to, or less than 0.80, any sudden disturbance would not reduce the unit shearing resistance below 2.1 kips per sq ft. Consequently, no momentary pore pressure would result as long as the quake does not increase p_1 beyond its initial value of 7.5 kips per sq ft. However, if the void ratio is 0.832, or slightly more, then the effective intergranular value of p_2 would be 2.00 kips per sq ft, the unit shearing resistance would be 1.55 kips per sq ft, and there would be a developed pore pressure of $7.5 - 5.5 = 2.0$ kips per sq ft. This amount of suddenly developed pore pressure at the toes of the cells would cause piping of the sand between El. + 35 and El. + 55 outside the cells.

In the direct or box shear tests, the angle of internal friction of the submerged sand varied from 28.3° to 38.3°, with corresponding initial void ratios of 0.85 and 0.62, respectively. Thus, a constant value of $\phi = 28°$ in the foregoing computations is not strictly accurate; but in view of the fact that, at best, the density in place of the sand can only be estimated, no refinements in such computations are warranted.

With reference to Fig. 49, the sand below El. + 55 in the area of the cells was deposited from dump barges and as many as 200 tons of sand were dumped in the mass at one time. The sand filling the cells was deposited hydraulically and in a manner to minimize both segregation and unbalanced lateral pressure. The backfill behind the sheet piling cells consisted of both sand and bank-run fill, both being predominantly stable granular materials. Option was given to deposit the backfill either hydraulically or by dumping.

Laboratory studies indicated that, by dumping, a void ratio not exceeding 0.80 could reasonably be expected. This in turn indicates that no suddenly developed pore pressure or other instability would be realized with sudden dis-

turbance for values of p_1 not appreciably exceeding 8 kips per sq ft. In those cells in which steel H-piles were driven for the crane tower foundations there can be no doubt that densification of sand, both in and below the cells, occurred to an extent such that there can be no likelihood of instability of the sand mass under earth shock. In all the cells timber templet piles were driven, nine in each cell, prior to driving the sheet piling and when the top elevation of dumped sand was at El. + 55. Then the sheet piles were driven around the templet piles. These operations tended to further increase the density of the sand below El. + 55. The cells were then filled hydraulically before backfilling. With the sand level within the cells at El. + 107, the lateral movement of sand within the cells was quite appreciable, tending to increase the density of the sand in the cells.

The use of sand in this project, and in the manner described, resulted in a very considerable reduction of cost below that estimated for filling by broken rock and other locally available materials which were considered to have greater stability. Earthquakes of minor intensity can serve only to increase the sand density and therefore also the structural stability. Such minor disturbances have already been realized. In the event of a major disturbance, approaching that of 1906 in intensity, it is problematical if extensive damage can occur; and, with the passage of time, the likelihood of any such damage is progressively lessened.

This project is of special interest for the reason that it invites consideration of possible control procedures in earthwork of this type and on this scale. The basic difficulty is that of obtaining undisturbed samples of sand on a scale required for a reliable survey of conditions. Perhaps the problem could be studied to better advantage by large-scale model tests. The results of research should have wide application and they would be especially applicable in the case of bulkheads with sand backfills.

CAVE-INS OF SANDY BACKFILLS

By J. C. Gebhard,[45] M. ASCE

Synopsis

The other papers in this program have dealt principally with various laboratory and design aspects of lateral earth pressures against bulkheads. In applying experimental and theoretical knowledge to the building of actual structures, the engineer is never quite sure just how conservative his designs and construction procedures have been unless there is some visible sign of distress or actual failure. Since such occurrences help to build up the fund of practical judgment, Commander Coxe and the writer reasoned that it would be of interest to discuss a few jobs that gave disappointing results. This paper concerns a case involving a nonplastic sandy backfill whereas the last Symposium paper by Commander Coxe describes a job that involved plastic soils.

Description

During World War II, it was necessary to increase the land area at one of the United States Naval stations by filling an indentation in the harbor with hydraulically placed sandy material. It was also necessary to construct a

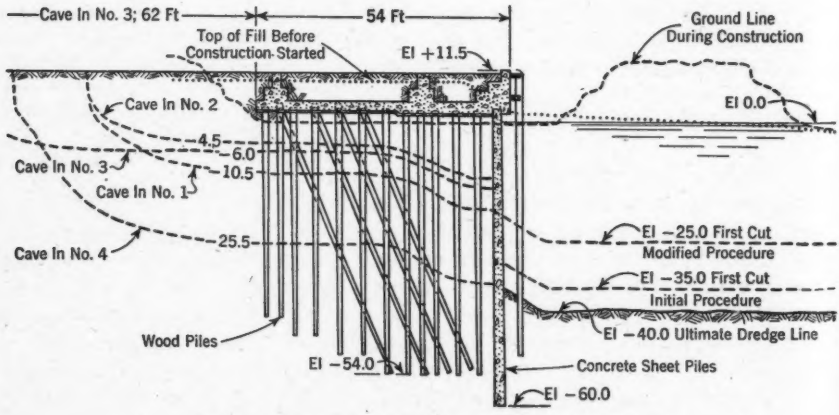


Fig. 53.—Typical Cross Section Through Quay Wall

3,000-ft quay wall across the outer end of the bight to retain the fill and to provide a wharf with a 40-ft depth of water alongside.

The quay wall (see Fig. 53) is of the relieving platform type with concrete sheet piles on the outer face. A flat slab concrete platform, about 53 ft wide,

[45] Capt. (CEC), U. S. Navy, Public Works Officer, 9th Naval Dist., Great Lakes, Ill.

with its bottom surface about 1.5 ft above low water is supported on a mat of closely spaced untreated wooden piles (see Fig. 54). The concrete sheet piles are anchored to the platform and the lateral thrusts are taken by wooden batter piles. A gravity wall extending to El. 11.5 forms the outer edge of the wharf and retains the surface fill.

The depth to the original sandy bottom varied considerably but was taken as El. −20 for purposes of design. Before starting work on the quay wall, a wide sand dike was constructed on the line of the quay wall to about El. + 11. The dike material was placed by a slotted dredge discharge pipe and it is reported that most of the thin overburden of muck or silt on the bottom of the bay was washed out in the process. Wooden test piles indicated that all piles would have to be jetted and that bearing values greater than 40 tons per pile would be reached at a point elevation of about El. −30. The fill is a fine nonplastic sand weighing about 110 lb per cu ft. All of it passes a No. 8 screen; about 90% passes a 50-mesh screen; 20% passes a 100-mesh screen; and only 4% passes a 200-mesh screen. The outboard beach of the dike was on a slope of about 1 on 10 prior to the beginning of the work on the quay wall.

Construction Procedure

The quay wall contractor first excavated a wide trench in the dike to about El. 1. He then jetted and drove the mat of wooden piles to a point elevation

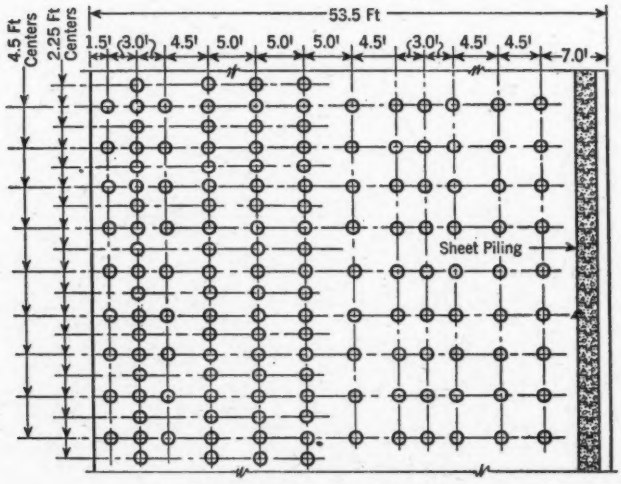


Fig. 54.—Typical Pile Plan

of about El. − 54. After this, he leveled off the top of the fill, placed a 4-in. layer of crushed stone, and cast all but the outer 9 ft of the concrete relieving platform. He then jetted and drove the concrete sheet piles to a point elevation of El. − 60 and anchored them back to the relieving platform. A general view of the construction is shown in Fig. 55.

At first it was planned to use steel sheet piles along the outer face of the wharf to retain the fill but, because of wartime conditions, they could not be obtained. Accordingly, concrete sheet piles, 24 in. deep and 19 in. wide, were

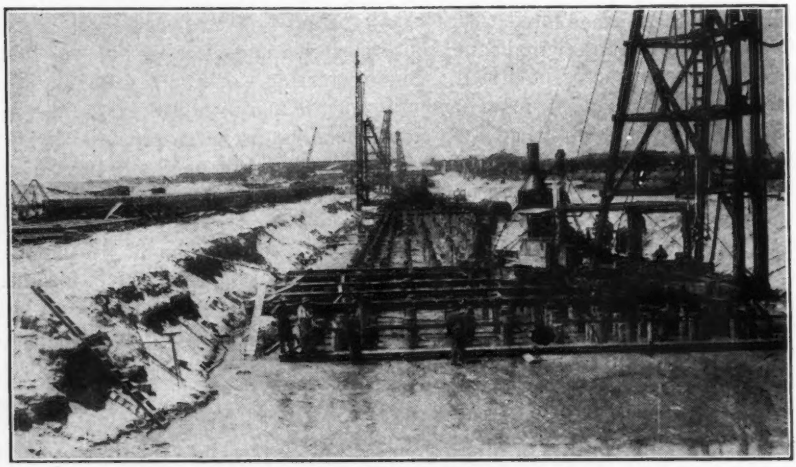
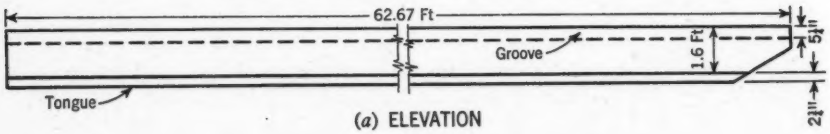
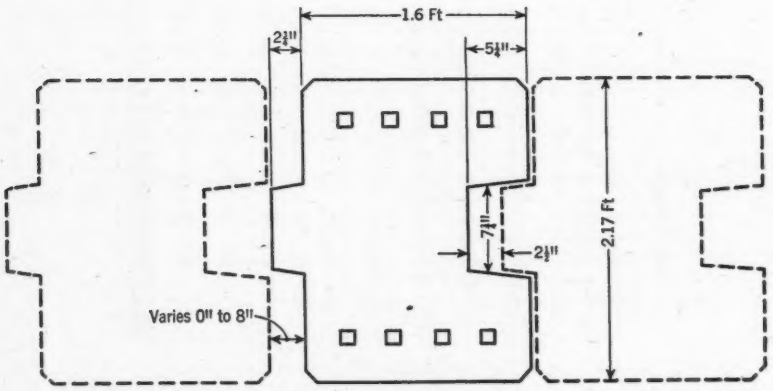


FIG. 55.—QUAY WALL UNDER CONSTRUCTION



(*a*) ELEVATION

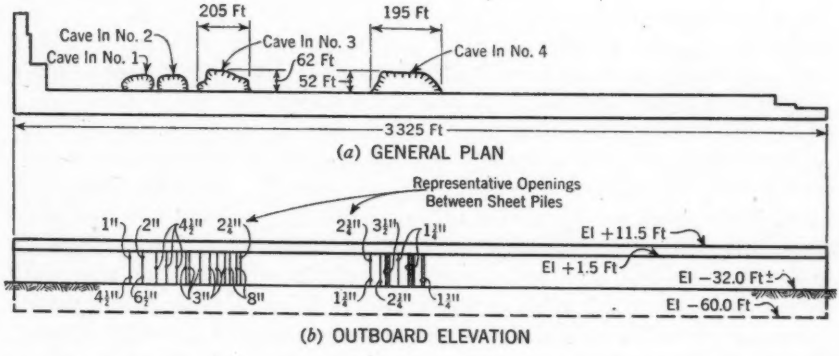


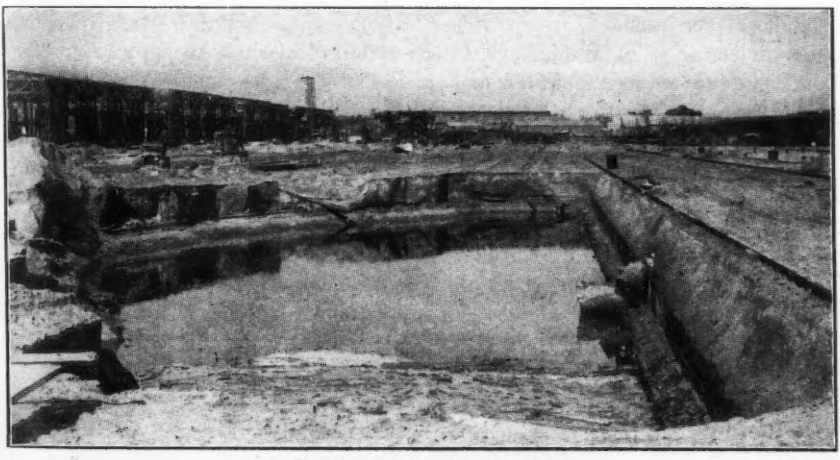
(*b*) SECTION

FIG. 56.—DIMENSIONS OF CONCRETE SHEET PILE

used. The tongues projected 2¾ in. and the grooves were 5¼ in. deep. Thus, vertical holes were left between the piles, measuring about 2½ in. by 7½ in. in plan as shown in Fig. 56. It was recognized that tight joints between the

sheet piles could not be expected in practice. The specifications, therefore, required that all joints be cleared of sand and silt, by a water jet, to a depth of 3 ft below the future dredge line—namely, to El. −43. The holes and spaces between the piles were then to be filled with cement grout pumped into place from El. −43 to the tops of the piles. To test the effectiveness of arrangement, two 30-ft sheet piles were driven so that their lower ends were about 10 in. apart with their tops in contact. Neat cement was forced into the groove





(c) VIEW OF CAVE-IN NO. 3

FIG. 57.—LOCATION OF CAVE-INS BEHIND QUAY WALL

through a pipe whose lower end was raised as the grout rose around it. When the piles were pulled, it was found that the grout had effectively filled the gap between the piles. Therefore, it was decided to proceed with the use of neat cement grouting. Because the entire lengths of the sheet piles were in the ground, it was not possible to inspect the results of driving or grouting until after the material on the outside had been dredged.

To speed his work, the dredging contractor began by dredging to El. −35 in one cut right to the sheet piles, using a 30-in. suction dredge. He then intended to make a second cut to reach the required depth of 40 ft. As the dredging progressed, four large cave-ins occurred suddenly soon after the outside material was removed. The location of these failures, and a view of cave-in No. 3 are shown in Fig. 57. These cave-ins extended as much as 60 ft beyond the inner edge of the relieving platform, or more than 100 ft from the sheet piles. They varied in depth to 35 ft and in length to about 200 ft. The largest single cave-in involved about 8,000 cu yd. It is impossible to determine the rate at which the material was lost since it is not known how long the loss may have been in progress under the relieving platform before the final cave-ins occurred. It appears however that as much as ±7,000 cu yd were lost in 24 hours, or an average of about 300 cu yd per hr. The peak rate is not known.

Inspections by divers showed that in about three hundred of the two thousand joints, the adjacent faces of the sheet piles did not meet properly by amounts varying from less than 1 in. to a maximum of about 8 in., measured at El. −30. These conditions were scattered along the entire length of the wall and there were numbers of openings extending through the wall where grout had failed to plug the joints effectively. In some cases, loose cylinders of grout the size of the grout pipe were found where the character of the bottom material had prevented proper intrusion of the grout.

The officer-in-charge was concerned about exposing the untreated wooden piles under the relieving platform to the action of marine borers and rot. Accordingly, he made a detailed exploration of the settlements that occurred under the entire length of the platform in locations other than the cave-ins and found that the fill had settled over wide areas from little or nothing to as much as 30 in. There was no evidence of any lateral translation of the relieving platform or of any part of the sheet piles. Mounds of sand found by divers outside the sheet piles had slopes as steep as 2 on 1. However, divers did not report seeing sand pouring through the openings. In one case, a hole to El. −44 was found outside the sheet piling in a region where the level of adjacent dredging was about 10 ft higher. It is not known whether this resulted from accidental overdredging or from a "blowout" under the ungrouted gaps between the piles below El. −43, or from both.

The dredging procedure in the second half of the job was then changed so that the first cut was made to El. −25, the second to El. −35, and the final cleanup cut to El. −40. Also, a small berm was left at the foot of the sheet piling to guard against overdredging, and the cutter head was kept at least 25 ft from the wall. The −25-ft level was used as a working platform from which divers grouted all suspicious looking joints. Wide joints that appeared dangerous were grouted practically to the tips of the piles before the dredging was extended to lower levels. With this revised procedure no further cave-ins occurred. The contractor finally pumped concrete sand under the slab so that at times the pavement back of the wall was heaved without, however, starting any further loss of material.

Discussion of Failure

It is not uncommon for a structurally adequate wall to fail to retain the backfill because of leaks through the sheet piling. Ordinarily, without the relieving platform, leaks can be detected early by the formation of small holes in the surface just behind the sheet piles. With the platform, there was no chance to see such holes if they formed. The cave-ins occurred suddenly and covered rather large areas. (In Fig. 53 the cave-in lines beneath the platform were assumed.)

The obvious answer, of course, is to make the sheet piles tight. Since this may not always be entirely practicable it is desirable to understand how material is lost through small openings, so that precautionary steps may be taken to provide appropriate safeguards. This appears to be especially important where sand dikes are relied on to reduce the lateral pressures of clay backfills, since a loss of dike material might then subject the bulkhead to dangerous lateral loads.

In this case a fine sand fill was placed hydraulically to its full height before any of the piles were driven. Large storehouses were built on spread footings on this type of fill with no noticeable settlement. Although the pile jetting operations probably loosened the sand, it is likely that the final driving of the last few feet of each pile, without jetting, recompacted it to some extent. No data are available to show what densities were actually obtained. Some of the sand that escaped was virgin bottom, not recent fill.

A number of explanations have been considered. A sudden jar caused by construction operations, or a sudden outward movement of the sheet piling, might cause a "quick" condition in the fine sand and make it behave temporarily as a heavy liquid mass. There was a suspicion that the cutterhead of the dredge struck the piling in one case. However, there is no suspicion of this in the other cases and no movements of the piles were observed. Therefore, it is not likely that the cutterhead of the dredge struck the piling since such an occurrence would not explain the fact that the loss of material continued for 24 hours. Another explanation considered was that, as sand slowly washed out between the sheet piles, cavities were formed behind the piles and these holes were maintained by the arching action of the sand. As the cavities enlarged and the loads become too great, the arches collapsed successively and built up pore pressures which liquefied the material and permitted it to flow out rapidly. The extent to which arched cavities could form would depend on the cohesion of the material in the backfill since ordinary cohesionless sand will not arch in this manner under water. This also is not a likely explanation even though such action may have occurred locally and contributed in a minor way to the loss of some material; nor is it likely that the phenomenon of internal erosion was involved—that is, the gradual formation of underground channels caused by percolating water escaping into a free opening through which material can be carried.

Where the grouting extended only a few feet below the dredged level, and the spaces between the piles were large enough, it is possible that the passive pressure on the outside was not great enough to resist the inside, active pressure.

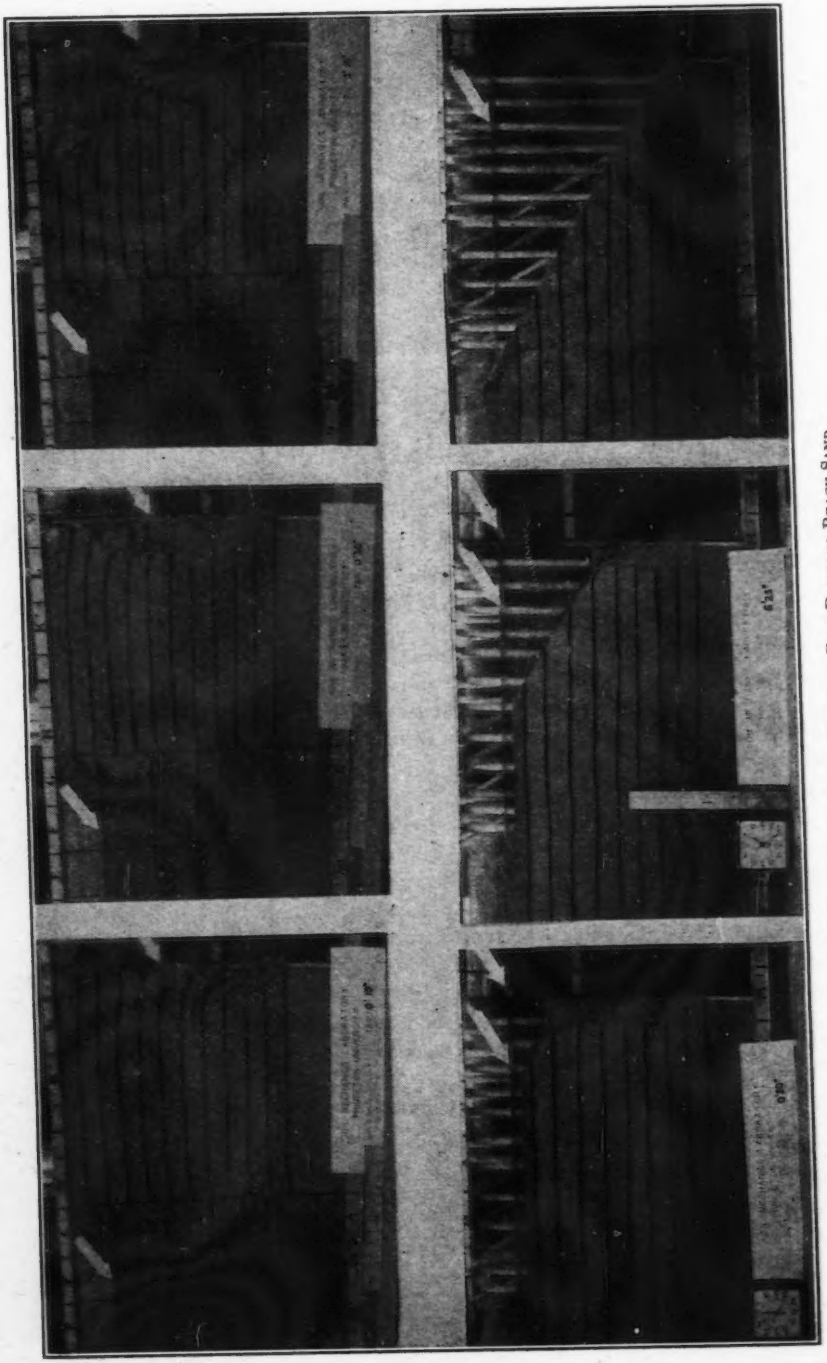


FIG. 58.—LABORATORY TESTS ON FINE DAYTONA BEACH SAND

This may have caused blowouts which then created quick conditions behind the bulkhead.

It is concluded, however, that most of the loss was started by the simple escape of cohesionless sand through openings between the piles above the dredged bottom. In an effort to visualize what might have happened, Professor Tschebotarioff ran a few laboratory demonstrations with a fine Daytona Beach sand. The tests were made in a glass sided tank 60 in. long by 24 in. high by 8 in. thick. A semicircular hole, 2 in. in diameter, was cut in the bulkhead against the glass side about 16 in. from the top.

In the first case the water was kept at the same level on both sides of the bulkhead and above the level of the sand. When the plug was removed, the cohesionless sand ran out of the hole by gravity. This quickly formed a vertical "pipe" of moving "quick" sand falling through water. The hole became enlarged within a few minutes. In the second case, the water was kept above the sand but was lowered on the outside. The action was the same but accelerated.

In the third case (see upper row, Fig. 58), water was kept slightly below the sand surface in back of the bulkhead and was kept somewhat lower on the outside. The action was similar to the other cases. In still another case (see lower row, Fig. 58), dowels were installed to simulate piles; again, the action was similar. The piles had little effect in retarding the rate of loss of the sand. It is possible that the reduction of the static pressure in the moving mass, as it acquires velocity, accelerates the loss of material from the banks, since the pressure in the banks would then be relatively greater. The escaping material was in a quick condition and the action would be expected to continue until the sand reached its natural angle of repose. In the prototype, it is entirely possible that chunks of material having some cohesion might have tumbled off the banks and caused a successive spread of localized quick conditions. Also, the wood piles could retard the loss of material having some cohesion and aid in building up higher and steeper banks than would otherwise be the case. This condition would tend to build up larger cavities and pressures and to produce sudden slides. The basic initial cause, however, appears to have been the simple flow of noncohesive material seeking its natural angle of repose under water.

The foregoing suggests that sandy materials used in backfills not only should have as great an angle of repose as practicable but also should be selected and placed so as to minimize the danger of loss from possible leaks in the bulkhead. The studies indicate the desirability of using densely graded coarse angular materials against the bulkhead to stop the loss of fines and to develop arching action across small openings. Of course, the thicker the sand dike behind the wall, the greater will be the factor of safety against damage resulting from unexpected leakage of the backfill.

Summary

Although there is nothing particularly new in this case, it emphasizes certain points—that is: (*a*) The importance of having tight walls sufficiently deep; (*b*) the need for guarding against conditions that can result in temporarily induced

pore pressures which, in turn, cause a loss of materials or excessive lateral pressures not contemplated in the design; (*c*) the fact that a dense mass of piles behind the bulkhead cannot be relied on to retard the loss of materials or to reduce lateral pressures especially if a quick condition is initiated; and (*d*) the material that is placed immediately in back of the bulkhead should be carefully selected and its placement controlled. Where a dike of selected material is relied on in the design to reduce the lateral pressures of other softer backfills, it is especially important to guard against its loss and to provide means for easy inspection and replacement of lost material. The Bureau of Yards and Docks now provides vertical 12-in. pipes between bents to facilitate inspection and replacing material under relieving platforms and pavements.

FAILURE OF QUAY WALL AT MARE ISLAND, CALIFORNIA

BY L. C. COXE,[46] ESQ.

The failure of the south quay wall at the Navy Yard, Mare Island, California, and subsequent investigations, conclusions, and remedial measures, are described in this paper.

The south quay wall, approximately 850 ft long, was part of a project to provide berthing and servicing facilities for the fleet. As shown in Fig. 59, the entire project included a north quay wall, a south quay wall, a west quay wall,

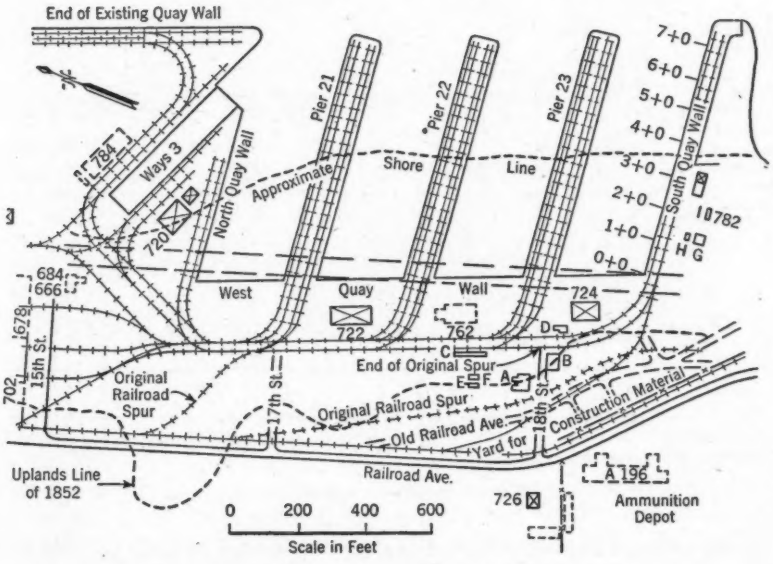


FIG. 59.—FACILITIES FOR BERTHING AND SERVICING SHIPS

and three finger piers. The south quay wall was constructed during the latter part of 1941 and the early months of 1942. Speed of construction was essential, and the rapid completion of all these waterfront facilities was the desperate need of the day.

The south quay wall is typical of many such structures constructed by the Civil Engineer Corps of the United States Navy. Reinforced concrete sheet piling 2 ft thick was used because of the shortage of steel. Behind the sheet piling, Fig. 60 shows a timber relieving platform supported on bents of twelve vertical wood piles and four wood batter piles. These bents, spaced at intervals of 5 ft, also support the heavy concrete foundations for the crane

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rails. A concrete head wall or quay wall, supported by the concrete sheet piling and the timber platform, completes the structure. Selected backfill was then placed on the relieving platform up to finish grade. Concrete anchors, supported by wood vertical and batter piles and connected to the quay walls by steel tie rods, 2 in. and 2.5 in. in diameter, were not included in the original design but were installed after failure of the wall. Fig. 61 is a longitudinal view of the quay wall during construction, indicating the size and weight of the crane railway girders and foundations.

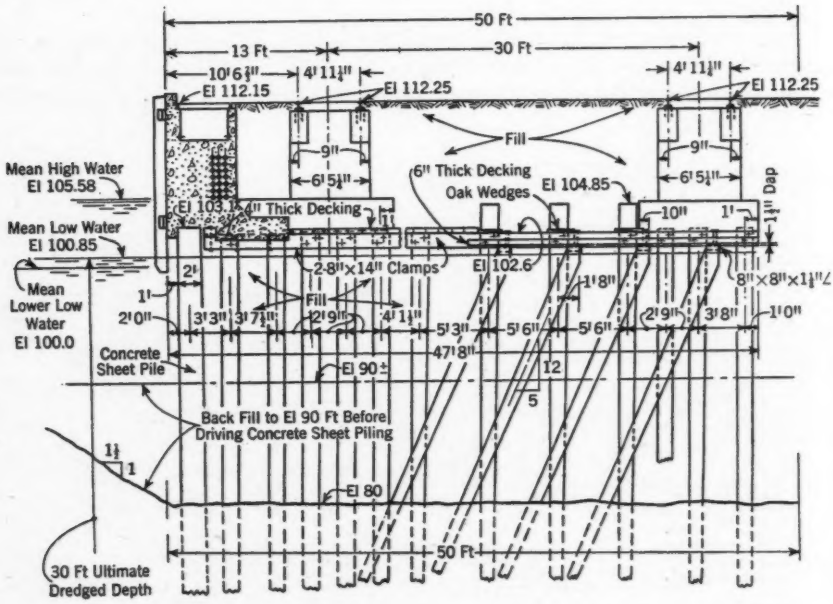


FIG. 60.—TYPICAL SECTION OF THE SOUTH QUAY WALL

Construction was started by using a suction dredge to clear the site of the quay wall to El. 70, or 30 ft below mean lower low water. For design purposes, the elevation of the mean lower low water was taken as El. 100. After dredging was completed, 20 ft of fill material were deposited in the quay wall area by bottom dump barges. Then the wood piling was driven for the relieving platform and framed with timber at the platform elevation (El. 102). The concrete sheet piling was driven from a derrick barge using a Vulcan No. 0 hammer. The concrete quay wall was then constructed to anchor the top of the sheet piling, and backfill was placed to the underside of the relieving platform deck. Finally, the relieving platform was constructed, craneway footings were poured, and the relieving platform was backfilled to El. 112.

Cracks began to appear in the face of the concrete quay wall in January, 1942, shortly after it had been poured (see Fig. 62). The relieving platform in the vicinity of the early cracks had not been loaded with backfill. In an attempt

to stop further outward motion of the wall, the relieving platform was immediately unloaded and 300 tons of "one-man" stone were deposited in front of the quay wall to stabilize the toe of the sheet piling. Construction of heavy pile-

FIG. 61.—CONSTRUCTION VIEW OF QUAY WALL

supported concrete anchorages was begun at Station 5 + 00 and at Station 7 + 25 (Fig. 59). By February, 1942, the anchorages were completed but not connected to the wall. On March 1, 1942, without the knowledge of the engineers in charge of the project, a sinking Navy tug was brought alongside the quay wall at Station 6 + 50 near its outer end and tied up pending salvage operations. The weight of this tug was further added to the lateral forces at work on the quay wall.

A survey made from January through May, 1942, revealed that, at Station 6 + 75, where the sinking tug was tied up, the wall had deflected outward a total distance of 6.4 ft, while settling vertically approximately 1.9 ft. Most of the vertical settlement and lateral movement of the wall occurred during January and February. Anchor rods were connected to the outboard anchor on February 24, 1942, and to the inboard anchor on March 4, 1942; yet despite these tie backs there was still an appreciable movement. Indeed, on later investigation, it was found that five of the tie rods, 2.5 in. in diameter, of the outboard anchorage had failed in tension. The failure of these tie rods was attributed to the additional load placed on the wall when the sinking tug was moored alongside.

The outward motion of the bulkhead caused five major cracks more than 1 in. wide and six minor cracks of less than 1 in. Fig. 63(*a*) shows an 8-in. crack at Station 6 + 69, at the location of the sunken tug and Fig. 63(*b*) shows two large cracks at the outboard end of the quay wall. Fig. 63(*c*) is a longitudinal view of the quay wall showing a crack and the irregularities in the face of the wall.

In July, 1942, O. J. Porter, M.ASCE, was called in to investigate the causes of the failure and to make recommendations on methods of arresting further movement of the quay wall. Soil borings were taken which revealed very soft materials to El. 17, or 83 ft below mean lower low water (see Fig. 64). Below this elevation, a stiff greenish sandy-clay stratum was encountered which contained considerable glauconite. This greenish glauconite clay is a marine deposit formed by the action of organic matter during a period when there was little or no filling. Beneath this glauconitic layer was a stratum of brown clay formed by oxidation and weathering of older shales. Shale bedrock, probably cretaceous, was encountered beneath the brown clay at approximately El. 8.

Above the glauconitic clay, the material consisted of relatively recent marine deposits of clay, peat, and mud. The fill material under the relieving platform was found to be very soft and unstable. This condition probably was the result of the slaking of the clay-shale material used in the filling operation. The material under the fill had a unit wet weight ranging from 87 lb per cu ft to 102 lb per cu ft and a moisture content ranging from 52% to 118% of the

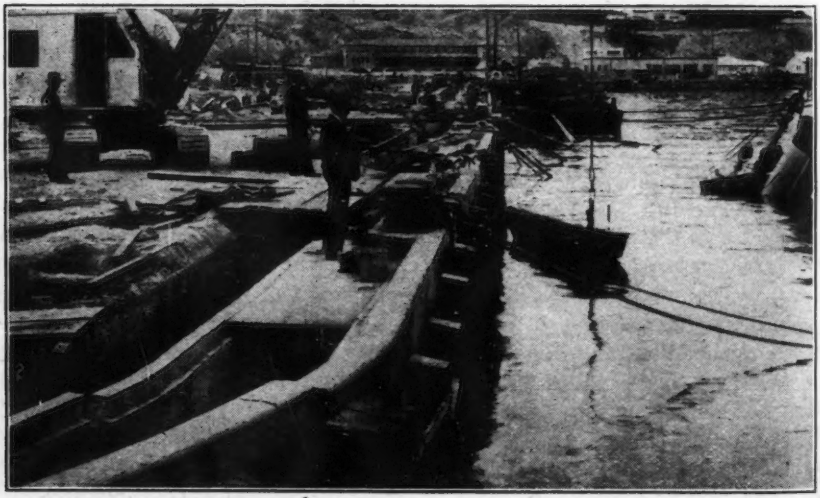


FIG. 62.—CONSTRUCTION PROGRESS, FEBRUARY, 1942; SUNKEN TUG IN RIGHT CENTER

dry weight of the soil particles. All indications were that the excessive lateral pressure acting on the bulkhead was caused by pore pressure, both in the backfill and in the fine-grained soil underlying the backfill. The pore pressure was caused by the weight of the 20 ft to 30 ft of fill placed during construction and

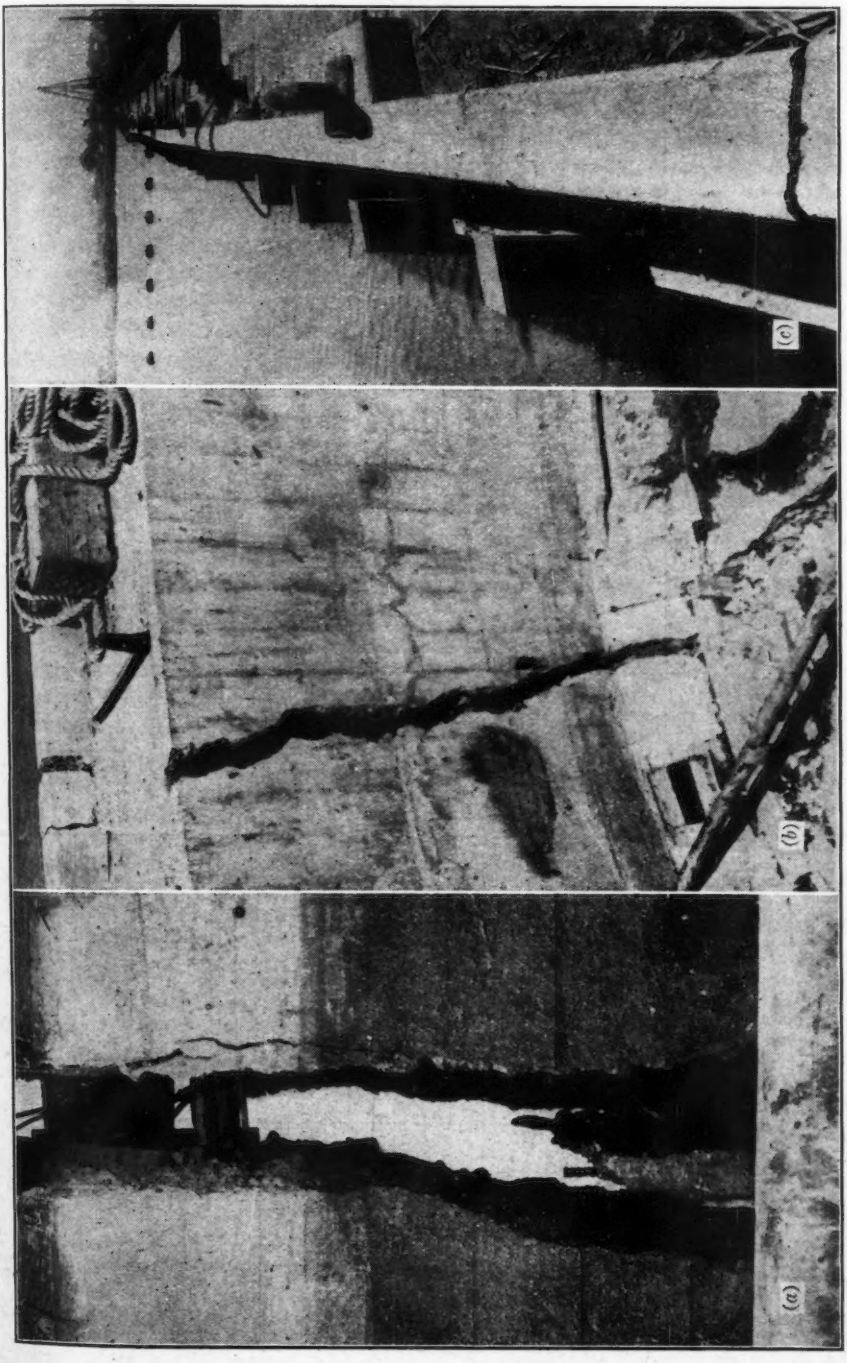


FIG. 63.—MAJOR CRACKS IN THE BULKHEAD

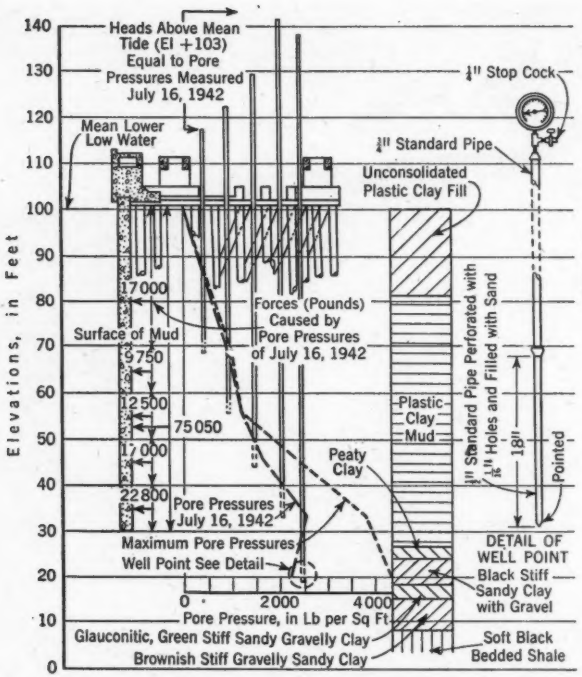


FIG. 64.—PORE PRESSURES

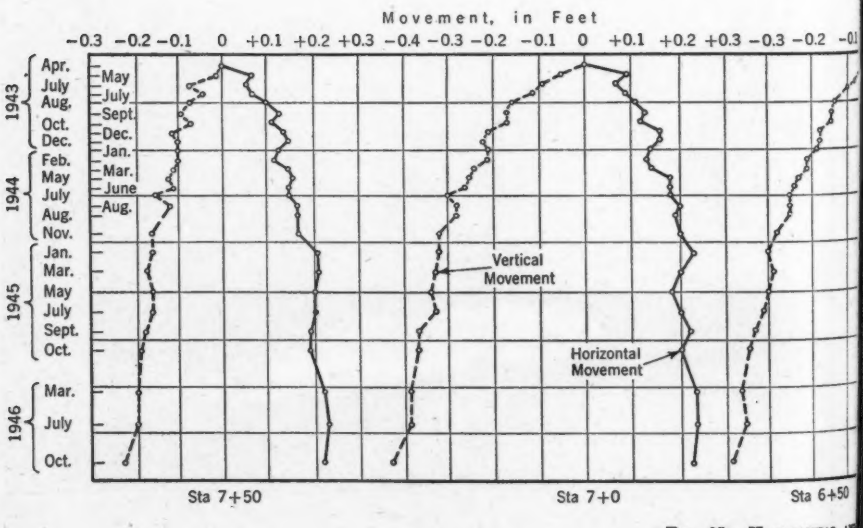
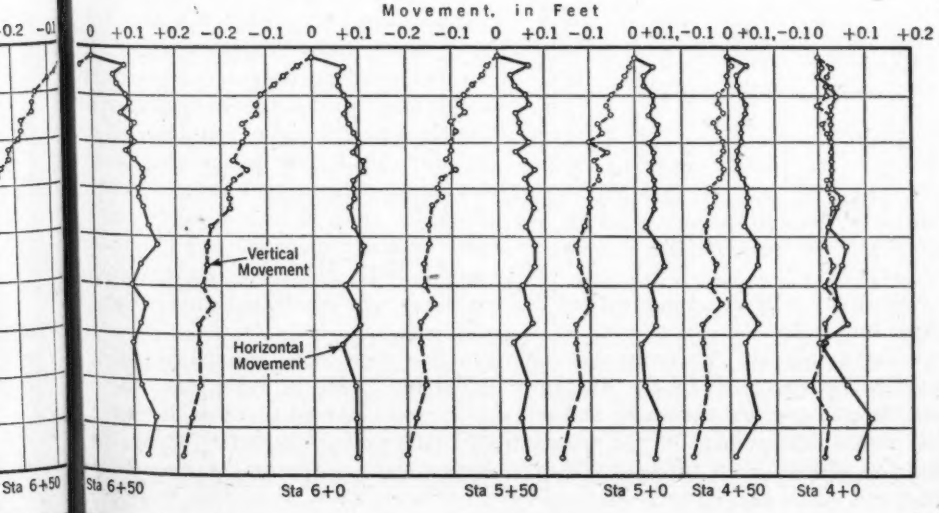


FIG. 65.—HORIZONTAL AN

also by that part of the weight of the quay wall structure which was transmitted as skin friction from the piles to the mud. To determine the extent and variation of pore pressure with depth, pore pressure measurements were made at El. 71, El. 56, El. 45, El. 33, and El. 20. A 1-in. soil sampler, devised by Mr. Porter, with a 0.5-in. perforated pipe extension was driven into the soil to the desired depth; and, after filling the pipe with water, pressures were measured by a gage attached to the upper end of the pipe. Readings were taken over a period of days and recorded as pounds per square inch after first correcting for the weight of the column of water above the tide level existing at the time the readings were taken. These readings indicated pore pressures ranging from zero at the surface to a maximum value of approximately 4,500 lb per sq ft 80 ft below the surface. The effect of pore pressure was to increase the lateral forces on the bulkhead by 75,000 lb per lin ft, over and above that which would have existed had pore pressure not been present. Pore pressure alone increased the lateral pressure 40% or more over that which might be calculated by assuming fully mobilized frictional and cohesive resistance in the soil.

To relieve this pore pressure quickly, work was begun in August, 1942, based on Mr. Porter's recommendation, to install vertical sand drains to a depth of 80 ft below mean lower low water. These drains were spaced 8 ft on centers in one direction and 10 ft on centers in the other, in the area under the relieving platform; and those in back of the relieving platform were spaced 12 ft by 13 ft. A total of three hundred and thirty-six drains were driven. The construction procedure called for the removal of sections of the relieving platform, cleaning out the mud and fill to a depth of 3 ft below the relieving platform, and backfilling with clean, coarse, concrete sand, or pea gravel. The sand drains were driven using a steel pipe, 12 in. in diameter, with a collapsible cap on the end. The mandrel was slowly withdrawn as the sand filled the driven hole. The



VERTICAL MOVEMENT

main difficulty was the refusal of the sand to stay in place when the mandrel was pulled. This difficulty was finally overcome by placing an airtight cap on the top of the mandrel after it was filled with sand and applying air pressure. The air pressure forced the sand out of the mandrel as the mandrel was withdrawn. The driving of the sand drains was completed in October, 1942.

In addition to the sand drains, and to hasten the reduction of pore pressure, surcharge in the form of sand blankets 5 ft or more thick was placed at various locations on the fill. When the pore pressures were reduced and the quay wall stabilized, the sand blankets were removed.

The action of the sand drains and surcharge in accelerating consolidation and pore pressure reduction was very effective although no actual data as to the reduction of pore pressures are available. This method of stabilization, combined with the anchored tie rods, completely checked any further motion of the quay wall and effective stabilization was realized in December, 1942—approximately two months after the sand drain installation was completed.

Inasmuch as the movement of the wall had stopped, it was decided in January, 1943, to dredge alongside the quay wall to the full 30-ft depth—necessitating the removal of 20 ft of clay from the outside face of the wall. The dredging was completed in February. Structural repairs to the pier were completed in May, 1943, and the quay wall was placed in service. Fig. 65 shows the results of the stabilization procedures. Elevations and alinement variations for the stations shown cover the 42-month period from April, 1943, to October, 1946. The minor changes of elevation and alinement occuring since May, 1943, were undoubtedly caused by the live load on the pier and the additional dredging. The changes in alinement for the 42-month period from April, 1943, to October, 1946, are on the order of only one twentieth of the magnitude of the changes during the 4-month period from January to May, 1942, prior to the installation of sand drains and anchors (see Table 3). The changes in elevation were similarly reduced. As of January 1, 1947, no further movement of the quay wall was observed and the backfill is now considered completely stabilized.

TABLE 3.—HORIZONTAL AND VERTICAL MOVEMENT, IN FEET

Station (see Fig. 57)	Outward Deflection		Change in Elevation	
	4-month period[a]	42-month period[b]	4-month period[a]	42-month period[b]
7+50...	4.0	0.22	−1.12	−0.30
7+00...	5.8	0.22	−1.96	−0.43
6+50...	6.2	0.13	−1.97	−0.38
6+00...	4.9	0.09	−1.45	−0.29
5+50...	4.5	0.06	−1.37	−0.20
5+00...	4.2	0.04	−1.18	−0.16
4+50...	3.8	0.01	−0.75	−0.08
4+00...	3.1	0.08	−0.30	+0.01

[a] 4 months from January, 1942, to May, 1942.
[b] 42 months from April, 1943, to October, 1946.

The account of this quay wall failure further emphasizes the many soils problems encountered by the Bureau of Yards and Docks in waterfront construction throughout the world, and explains the great interest that this Bureau has in the development of test procedures for determining, accurately, characteristics of soils of all types, methods of design, and construction procedures that will insure against future failures.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

CONTINUOUS FRAME ANALYSIS BY ELASTIC SUPPORT ACTION

Discussion

BY RALPH W. STEWART, AND MAURICE BARRON

RALPH W. STEWART,[51] M. ASCE.—A method is presented in this paper which is efficient in computing the bending moments in complex continuous frames such as Vierendeel trusses, and which is as accurate as the common theory of flexure. This gives the paper considerable value as a reference.

In developing the method by applying certain old theorems of structural mechanics, it appears to the writer that the preliminary work is cumbersome. For example, in order to obtain Eq. 14*a* the authors show five drawings (Figs. 4(*a*) to 4(*e*), inclusive). By simply adding two lines to Fig. 4(*b*) and transferring to it the longitudinal dimensions as shown in Figs. 4(*c*), 4(*d*), and 4(*e*), it is converted into Fig. 23, which shows all the elements of the flexure and makes the other four drawings unnecessary. The authors' procedure for deriving Eq. 14*a* can now be duplicated term by term by referring to Fig. 23 and using its geometrical properties. This reduces the labor involved in the derivation and also eliminates the use of the conjugate beam theorem. Similarly, by adding two lines to each of Figs. 2(*b*), 3(*b*), and 5(*b*), reference to the conjugate beam theory and also the moment-area drawings can be omitted.

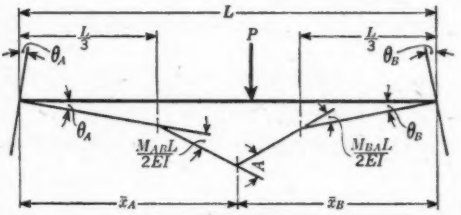


FIG. 23

Beginning with Clapeyron's discovery of the three-moment theorem, various special formulas or constants have been proposed for facilitating the analysis of stresses in continuous structures. These include, among others, the slope

NOTE.—This paper by J. Charles Rathbun and C. W. Cunningham was published in April, 1947 *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1947, by L. J. Mensch, Frederick S. Merritt, I. Oesterblom, Thomas C. Kavanagh, A. Floris, and Tao King; and November, 1947, by Leroy A. Beaufoy, A. A. Eremin, Robert B. B. Moorman, Eduardo Agramonte, Stephen J. Fraenkel and Robert L. Janes, and Phil M. Ferguson.

[51] Bridge Engr., City of Los Angeles, Los Angeles, Calif.

deflection equations, the conjugate beam theorem, the carry-over and special stiffness factors used in the end-moment distribution method, and the transmission coefficients. The authors now offer the "elastic length" and the "elastic factor" to add to the list. These special formulas or constants are derivatives obtained by combining part or all the six basic constants of flexure in one form or another.

Because the propriety of using the adjective "basic" in identifying the constants about to be described has been questioned,[52] it is necessary to explain why this is done. A suitable explanation can be presented by chemical analogy. The most important discovery in chemistry prior to atomic fission was that all molecules could be broken down into components (atoms) which could not at that time be further subdivided. Combinations of atoms became known as "derivatives"; for example, CH_4, C_2H_2, and C_3H_{18} are called hydrocarbon derivatives, and hydrogen and carbon may be considered as their basic components. A similar condition exists in machines, for which the basic components may be listed as shafts, gears, keys, pulleys, belts, and the like.

All conditions of flexure may be reduced to an assembly of the following constants, which are designated as basic:[53]

(1) The moment necessary to produce a unit change in the direction of the elastic curve of a beam when applied at the left end, the right end being hinged;
(2) The moment necessary to produce a unit change in the direction of the elastic curve of a beam when applied at the right end, the left end being hinged;
(3) The abscissa of the intersection of the end tangents to the elastic curve of the beam (or the center of gravity of the $\frac{M}{E_c I}$-diagram) for condition (1); and
(4) The corresponding abscissa for condition (2).

TABLE 19.—TABULATION

Line	Description	Factor	Columns				
			BA	DC	FE	HG	JI
1	Moments due to FEM in span GE	16/4,462	−0.03	+0.10	−0.34	+2.51	−0.85
2	Moments due to unbalance at point E	4/830.8	−0.04	+0.13	−0.45	+0.13	−0.04
3	Moments due to FEM in span CE	12/4,462	+0.63	−1.89	+0.26	−0.07	+0.02
4	Balanced fixed-end moments	....	....	....	....	....	....
5	Final moments (without sidesway)	....	+0.56	−1.66	−0.53	−2.57	−0.87

These four constants are known as the beam constants. They are interrelated so that, if three are given, the fourth can be computed by Maxwell's theorem of reciprocal deflections (angular). Constants (1) and (2) are the same as the term preceding the parentheses in any slope deflection equation.

[52] *Transactions*, ASCE, Vol. 108, 1943, p. 1230.
[53] "The Mechanics of Creep for Structural Analysis," by Joseph Marin, *ibid.*, p. 477.

The two additional basic constants, known as the load constants, are:

(5) The total curvature in the beam (equal to the area of its $\frac{M}{EI}$-diagram), when it is considered as simply supported and subjected to the external loading, which acts on it between the supports; and

(6) The abscissa of the intersection of the end tangents to the elastic curve for condition (5).

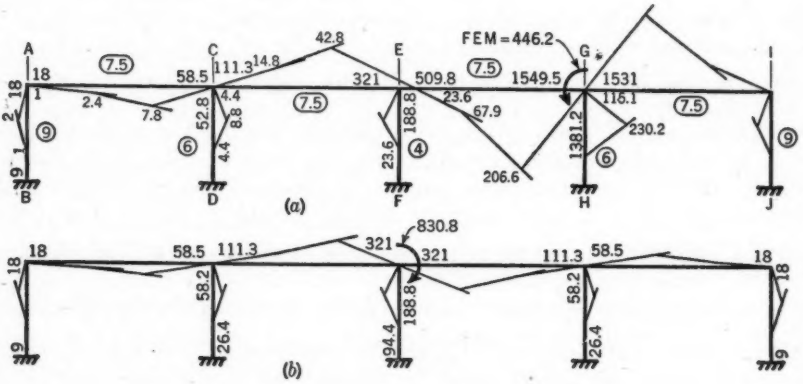


FIG. 24.—MOMENTS FOR EXAMPLE 3

Although constants (1) and (2), which are called the basic stiffnesses, can be further subdivided into force times distance, neither force (which might be axial), nor distance acting alone can produce flexure. The simplest condition which can produce flexure is a combination of force and distance; therefore, constants (1) and (2) have been reduced to the lowest possible terms as far as direct relationship to flexure is concerned.

OF MOMENTS, EXAMPLE 6

JOINT A		JOINT C			JOINT E			JOINT G			JOINT I		Line
AB	AC	CA	CD	CE	EC	EF	EG	GE	GH	GI	IG	IJ	
−0.06	+0.06	+0.21	+0.19	−0.40	−1.15	−0.68	+1.83	+5.55	+4.95	−10.50	+1.69	−1.69	1
−0.09	+0.09	+0.28	+0.25	−0.53	+2.45	−0.91	−1.54	−0.53	+0.25	+0.28	+0.09	−0.09	2
+1.27	−1.27	−4.12	−3.71	+7.93	−1.37	+0.51	+0.86	+0.30	−0.14	−0.16	−0.05	+0.05	3
....	....	....	....	....	−12.00	....	+12.00	....	....	....	....	....	4
+1.12	−1.12	−3.63	−3.27	+7.00	−12.07	−1.08	+13.15	+5.32	+5.06	−10.38	+1.73	−1.73	5

If a quantic, such as the stiffness value of a member as defined for use in end-moment distribution, contains more than one of these basic constants, it is a derived function and not a basic constant. With these basic constants (Fig. 23) well in mind, a computer does not need to burden his tool kit with extra items like the conjugate beam theorem.

The authors' Example 3 (Fig. 8) is easily solved by the application of only four of these constants, as illustrated by Fig. 24 and Table 19. In Fig. 24 all

angles of flexure are at the third points of the members. This solution involves a minimum number of slide rule settings and requires no preliminary computation of special constants. It may be assumed that the authors presented problems like Example 3 for the purpose of illustrating the use of the "elastic length" and the "elastic factor," rather than to shorten the solutions by using these special functions.

The authors' solution of Example 3 (Fig. 8), shows that $\frac{2I}{L}$ was used for the relative stiffness values of the members (constants (1) and (2) in the list of basic constants of flexure with E omitted). In Fig. 24, a set of values equal to $\frac{12I}{L}$ was used in order to avoid fractional values as much as possible. In Fig. 24, the series of angle values will remain identical for any proportional set of stiffness values but the moment values will vary directly as the stiffness values. The authors' "stiffness factors" can therefore be taken from Fig. 24. For example, the authors' stiffness factor at end E of member CE is $\frac{3.21}{6 \times 23.6} = 2.267$, which checks the value in Table 4. If Fig. 24 had been constructed by using $\frac{2I}{L}$ values for the relative member stiffnesses, the reduction factor 6 would disappear from this computation and the authors' elastic factors and stiffness factors could be obtained by simply dividing the end slope angles at the activated ends of the members by the moments at these activated ends, and vice versa.

It becomes troublesome to describe flexural analysis procedures with the words "stiffness" or "stiffness factor" meaning any one of the three following separate properties:

(1) Basic stiffness: Used in the slope deflection and traverse methods.
(2) End slope stiffness of a member with the opposite end fixed: Used in end-monent distribution.
(3) End slope stiffness of a member or an assembly of members for whatever remote end conditions exist: Used by the authors of this paper. It is the reciprocal of the "elastic factor."

Figs. 6, 7, and 8 are so easily solved by the use of basic constants that the use of the authors' special devices for such problems seems inadvisable. However, the solution of the Vierendeel truss shown in Fig. 12 is not easy by any method, and for structures of this class the authors' method appears to be useful.

MAURICE BARRON,[54] M. ASCE.—The value of this paper lies in the discussion which it will provoke. The authors are to be commended for bringing to the attention of the profession the fact that a method of direct moment distribution is possible, and that it is no longer necessary to use the methods of successive approximation. Unfortunately, the proposed method is elementary when compared to more advanced methods of direct moment distribution.

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One such method[55] is that developed by Harris Epstein. Another method,[56] somewhat similar to the Epstein method, has been fully developed by R. C. Brumfield, M. ASCE. It appears that many of the refinements of the Brumfield method could be incorporated into the authors' method. For instance, their concept and use of the artificial fixed joint is evidence of a "stumbling block" that Professor Brumfield has ingeniously eliminated. This problem is encountered in every re-entrant frame. The difficulty arises from the fact that there is no starting point of known restraint. The authors, therefore, fix as many joints as are required and later release these joints, with a further corrective moment distribution. Professor Brumfield has developed a method whereby he can "jump into" any such re-entrant frame and very accurately estimate the transmission coefficient necessary to complete the analysis and, at the same time, check the estimated values. This means that a starting point is available and no corrective distribution is required because of arbitrary fixing of joints. Furthermore, this becomes a very useful tool in analyzing an individual member of a complicated frame without making an elaborate analysis of the surrounding members. The problem of sidesway, or the adjustment for unbalanced shears, is solved by very much the same procedure in both methods.

In order to illustrate the relatively weak points of the authors' paper the writer has used the Brumfield method to solve Examples 4 and 5. Additional symbols necessary for adaptation of this method to the authors' examples include:

The flexibility factor, Q, similar to the reciprocal of the ordinary stiffness factor, for uniform prismatic members measured by L/I; the transmission coefficient, C, similar to the ordinary carry-over factor and denoting the ratio of the moment arriving at one end of a member to the moment applied at the opposite end of the same member; the stiffness index, i_s, which is a measure of the resistance a member offers to the rotation of the joint to which it is attached (each member has two stiffness indexes, one for each end); and the restraint index, R, of a member radiating from a joint, a measure of the resistance to rotation offered to that member when it is used to actuate the joint by the other members radiating from the same joint. The relations between these factors are expressed by:

$$i_s = Q\,(2 - C) \qquad (87)$$

in which the "going away" transmission coefficient is

$$C = \frac{1}{2 + \dfrac{R}{Q}} \qquad (88)$$

and

$$\frac{1}{R} = \frac{1}{i_{s1}} + \frac{1}{i_{s2}} + \frac{1}{i_{s3}} + \cdots + \frac{1}{i_{sn}} \qquad (89)$$

[55] "Standards of Design for Concrete," *No. 5Yb*, Bureau of Yards and Docks, U. S. Navy Dept., November 15, 1929, pp. 148–177.

[56] "Moving Loads on Restrained Beams and Frames," by R. C. Brumfield, *Transactions*, ASCE, Vol. 111, 1946, p. 805.

angles of flexure are at the third points of the members. This solution involves a minimum number of slide rule settings and requires no preliminary computation of special constants. It may be assumed that the authors presented problems like Example 3 for the purpose of illustrating the use of the "elastic length" and the "elastic factor," rather than to shorten the solutions by using these special functions.

The authors' solution of Example 3 (Fig. 8), shows that $\frac{2I}{L}$ was used for the relative stiffness values of the members (constants (1)) and (2) in the list of basic constants of flexure with E omitted). In Fig. 24, a set of values equal to $\frac{12I}{L}$ was used in order to avoid fractional values as much as possible. In Fig. 24, the series of angle values will remain identical for any proportional set of stiffness values but the moment values will vary directly as the stiffness values. The authors' "stiffness factors" can therefore be taken from Fig. 24. For example, the authors' stiffness factor at end E of member CE is $\frac{3.21}{6 \times 23.6} = 2.267$, which checks the value in Table 4. If Fig. 24 had been constructed by using $\frac{2I}{L}$ values for the relative member stiffnesses, the reduction factor 6 would disappear from this computation and the authors' elastic factors and stiffness factors could be obtained by simply dividing the end slope angles at the activated ends of the members by the moments at these activated ends, and vice versa.

It becomes troublesome to describe flexural analysis procedures with the words "stiffness" or "stiffness factor" meaning any one of the three following separate properties:

(1) Basic stiffness: Used in the slope deflection and traverse methods.
(2) End slope stiffness of a member with the opposite end fixed: Used in end-moment distribution.
(3) End slope stiffness of a member or an assembly of members for whatever remote end conditions exist: Used by the authors of this paper. It is the reciprocal of the "elastic factor."

Figs. 6, 7, and 8 are so easily solved by the use of basic constants that the use of the authors' special devices for such problems seems inadvisable. However, the solution of the Wierandeel truss shown in Fig. 12 is not easy by any method, and for structures of this class the authors' method appears to be useful.

MAURICE BARRON,[14] M. ASCE.—The value of this paper lies in the discussion which it will provoke. The authors are to be commended for bringing to the attention of the profession the fact that a method of direct moment distribution is possible, and that it is no longer necessary to use the methods of successive approximation. Unfortunately, the proposed method is elementary when compared to more advanced methods of direct moment distribution.

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One such method is that developed by Hardy Cross. Another method, somewhat similar to the Cross method, has been fully developed by R. C. Brumfield, M. ASCE. It appears that many of the refinements of the Brumfield method could be incorporated into the Cross method. For instance, their common use of the tentative fixed points as devices of a stimulating block that Brumfield has ingeniously eliminated. This problem is encountered in every re-entrant frame. The intuitive basis is the fact that there is a starting point of known restraint. The notation is, therefore, fix as many points as are required and later release the points, with further corrective movements distributed. Professor Brumfield has developed a method whereby the restraint in any such re-entrant frame may very accurately estimate the transmission coefficient necessary to complete the analysis and, at the same time, check the restraint values. It assumes that a starting point is available and no corrective distribution is required because of arbitrary fixing of points. Furthermore, it becomes a very useful tool in analyzing an individual member of a completed frame without making an elaborate analysis of the surrounding members. The problem of sidesway, or the adjustment for unbalanced stresses, is solved by very much the same procedure in both methods.

In order to illustrate the relative merits of the two methods, the writer has used the Brumfield method to solve Example No. 5. Additional symbols necessary for adaptation of this method to the two examples include:

The flexibility factor, Q, similar to the reciprocal of the ordinary stiffness factor, for uniform prismatic members means $Q = 5\ l/t$; the transmission coefficient, C, similar to the carry-over factor, and denoting the ratio of the moment arriving at one end of a member to the moment applied at the opposite end of the same member, the restraint index, r, which is a measure of the resistance a member offers to the rotation of the joint to which it is attached (each member has two restraint indexes, one for each end), and the restraint index, R, of a member radiating from a joint, a measure of the resistance to rotation offered to that member when it is used to balance the joint by the other members radiating from the same joint. The relations between these factors are expressed by:

$$r_n = Q\,(2 - C) \qquad (87)$$

in which the "going away" transmission coefficient is

$$C = \frac{1}{2 + \frac{R}{Q}} \qquad (88)$$

and

$$\frac{1}{R} = \frac{1}{r_{ab}} + \frac{1}{r_{ac}} + \frac{1}{r_{ad}} + \frac{1}{r_{ae}} + \cdots + \frac{1}{r_{an}} \qquad (89)$$

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[1] "Handbook of Design for Shipbuilding," Navy Dept., Bureau of Yards and Docks, U. S. Navy Dept., November 15, 1935, pp. 1345–1347.

[2] "Moment Tensile Connections for Steel Construction," by R. C. Brumfield, *Transactions*, ASCE, Vol. 111, 1946, p. 1000.

In Example 4 (Fig. 9), the authors have arbitrarily chosen joint B as a fixed joint, thereby making the carry-over factors toward joint B of members AB and DB equal to 0.500. The transmission coefficient corresponding to the first of these two carry-over factors can be estimated as follows: $C_{AB} = \frac{1}{2 + \frac{Q_{BD}\,(2 - 0.25)}{Q_{AB}}} = \frac{1}{2 + \frac{2\,(2 - 0.25)}{2}} = 0.266$. The approximate coefficient in the denominator is the average "going away" coefficient of all the restraining members at the joint toward which the desired estimated coefficient is transmitting.

In the computation for the estimated value of C_{AB}, only a rough estimate is required for the value of C_{BD} (= 0.25) which appears in the denominator. The value of 0.25 was chosen because the member BD is transmitting toward member DC, which is more flexible than member BD. If these two members were of equal flexibility, the value of 0.3 might be used, whereas if joint D had several restraining members, or one of great rigidity, the value of 0.35 might be used to estimate C_{AB}.

Similarly, C_{BA} is estimated to be: $C_{BA} = \frac{1}{2 + \frac{Q_{AC}\,(2 - 0.2)}{Q_{BA}}} = \frac{1}{2 + \frac{1.6\,(2 - 0.2)}{2}}$ = 0.291. The value of 0.2, as an approximation of C_{AC}, is used because member AC is transmitting toward the more flexible member CD. If the value 0.25 had been used, C_{BA} would equal 0.294; thus, a 25% change in the estimated coefficient results in only 1% change in the final coefficient, indicating that a crude guess is sufficient for very accurate results. The two estimated coefficients are entered in the tabulated computation of Table 20, and all the

TABLE 20.—COMPUTATION OF TRANSMISSION COEFFICIENTS AND TORQUE SPLITS, EXAMPLE 4

Joint	Member	Stiffness Index, i_s		Restraint index, R	Transmission Coefficient		Authors' carry-over factor	Torque split[a]
		Computation	Value		Symbol	Value		
A	BA	2.0 (2 − 0.266)[b]	3.47	2.89	C_{BA}	0.290	0.290	0.454
	CA	1.6 (2 − 0.194)	2.89	3.47	C_{CA}	0.240	0.258	0.546
C	DC	3.0 (2 − 0.317)	5.05	2.82	C_{DC}	0.340	0.341	0.358
	AC	1.6 (2 − 0.240)	2.82	5.05	C_{AC}	0.194	0.195	0.642
D	BD	2.0 (2 − 0.270)	3.46	4.98	C_{BD}	0.223	0.223	0.410
	CD	3.0 (2 − 0.340)	4.98	3.46	C_{CD}	0.317	0.333	0.590
B	AB	2.0 (2 − 0.291)[b]	3.42	3.55	C_{AB}	0.265	0.500	0.510
	DB	2.0 (2 − 0.223)	3.55	3.42	C_{DB}	0.270	0.500	0.490

[a] $R/(R + i_s)$. [b] Estimated coefficients.

transmission coefficients are computed, including the exact values of C_{AB} and C_{BA}. It is interesting to note the agreement between estimated and final coefficients. Also, a comparison of the Brumfield transmission coefficients and the authors' carry-over factors demonstrates that the two are identical at joints twice removed from the fixed joint, showing a rapid convergence to the true value.

The frame characteristics for Example 4 are found to be as shown in Fig. 25, from which the end moments in member AC, with joint translation prevented, are determined as follows:

$$M_{AC} = -\frac{[(2k - 3k^2 + k^3) - C_{AC}(k - k^3)]\,P\,L\,C_{CA}}{1 - C_{AC}\,C_{CA}}$$

$$= -\frac{[(2 \times \tfrac{1}{2} - 3 \times \tfrac{1}{2}^2 + \tfrac{1}{2}^3) - 0.194\,(\tfrac{1}{2} - \tfrac{1}{2}^3)]\,4 \times 24 \times 0.240}{1 - 0.194 \times 0.240}$$

$$= -7.30 \text{ kip-ft; and}$$

$$M_{CA} = -\frac{[(k - k^3) - C_{CA}(2k - 3k^2 + k^3)]\,P\,L\,C_{AC}}{1 - C_{CA}\,C_{AC}}$$

$$= -\frac{[(\tfrac{1}{2} - \tfrac{1}{2}^3) - 0.240\,(2 \times \tfrac{1}{2} - 3 \times \tfrac{1}{2}^2 + \tfrac{1}{2}^3)]\,4 \times 24 \times 0.194}{1 - 0.240 \times 0.194}$$

$$= -5.57 \text{ kip-ft.}$$

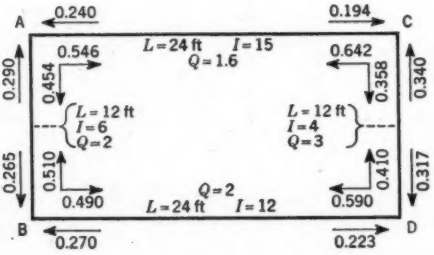


FIG. 25.—FRAME CHARACTERISTICS FOR EXAMPLE 4 (FIG. 9)

Distribution of moments in the frame is shown in Fig. 26, in which arrows to the right indicate positive moments (authors' convention), and those to the left indicate negative moments. This sign convention for moments has been used throughout the writer's solution of Examples 4 and 5.

The sidesway, or unbalanced shear problem, has also been worked out by the Brumfield method. From Fig. 26(*a*), the total unbalanced moments equal −1.87, which is twelve times the unbalanced shear. The moment corresponding to this unbalanced shear, from Fig. 26(*b*), is +1,879 *M*. Equating these two values, 1,879 *M* = − 1.87, from which 1,000 *M* = − 0.996. Application of this factor as a sidesway correction (lines 4 and 9, Fig. 26(*a*)) to the moments for no sidesway (lines 3 and 8) gives the final moments (lines 5 and 10).

In Example 5 (Fig. 10), the authors again use the expedient of fixing joints —in this case joint B. After the preliminary distribution, joint B is released and a corrective distribution is computed. The correction for unbalanced shears in this problem is more complicated than in the previous one, because there is not only a sidesway, but also a vertical sway. The authors' procedure in step (*g*) is to deform the structure a distance, $\Delta = 24/E$, and then to find the induced moments resulting from this distortion. Eight moments are thus required for distribution.

Line	Description	AB		AC		CA		CD	
1		0.14	7.30	7.30	0.14	0.15	5.57	5.57	0.15
2			0.04	0.04			0.03	0.03	
3	Uncorrected moment		7.20	7.20			5.45	5.45	
4	Sidesway correction	0.51			0.51		0.45	0.45	
5	Final moment		6.69	6.69			5.90	5.90	
				(a)					
		BA		BD		DB		DC	
6		0.48	1.93	1.93	0.48	0.43	1.77	1.77	0.43
7			0.01	0.01					
8	Uncorrected moment		1.46	1.46			1.34	1.34	
9	Sidesway correction	0.48			0.48		0.43	0.43	
10	Final moment (kip-ft)		0.98	0.98			1.77	1.77	

		1,000 *M*						667 *M*	
11		AB		AC		CA		CD	
12		454 AB[a]	3 AC	546 AC	2 AB	106 AC	9 AB	9 AB	106 AC
13		2 AB	9 BD	3 AC	148 BA	1 AC	29 BA	29 BA	1 AC
14		148 BA	103 CA	9 BD	1 BA	37 BD	1 CD	1 CD	37 BD
15		1 BA	31 DB	103 CA	6 CD	428 CA	93 DC	239 CD	2 CA
16		6 CD		31 DB	22 DC	2 CA		93 DC	6 DB
17		22 DC				6 DB			
18			513 *M*	513 *M*		448 *M*			448 *M*
				(b)					
19		1,000 *M*						667 *M*	
		BA		BD		DB		DC	
20		120 AB	9 AC	9 AC	120 AB	34 AC	27 AB	27 AB	34 AC
21		1 AB	2 BD	2 BD	1 AB	109 BD	9 BA	9 BA	109 BD
22		2 BA	27 CA	27 CA	2 BA	1 BD	76 CD	76 CD	1 BD
23		20 CD	106 DB	106 DB	6 DC	6 CA	1 DC	1 DC	6 CA
24		6 DC	1 DB	1 DB	20 CD	2 DB			2 DB
25		510 BA		490 BD		393 DB		274 DC	
26			486 *M*	486 *M*		432 *M*			432 *M*

[a] Letters following moment values indicate the joint of origin.

FIG. 26.—MOMENT COMPUTATIONS FOR EXAMPLE 4
(*a*) Final Moments
(*b*) Sidesway Distribution

In the Brumfield procedure (Table 21 and Figs. 27, 28, and 29) member AB is translated vertically downward with no rotation of joints. Since the translation for point A of member AE and AC is the same, and the Q-values and L-values are identical, the induced fixed-end moments, M_{EA}, M_{AE}, M_{AC}, and

TABLE 21.—COMPUTATION OF TRANSMISSION COEFFICIENTS, MOMENT SPLITS, AND TORQUE SPLITS, IN EXAMPLE 5

Joint	Member	Stiffness Index, i_s		Restraint index, R	Transmission Coefficient		Authors' carry-over factor	Moment split[a]			Torque split[b]
		Computation	Value		Symbol	Value					
F	EF	1.33 (2 −0.252)[c]	2.33	3.30	C_{EF}	0.224	0.235	....	....	....	0.586
	BF	2.0 (2 −0.351)	3.30	2.33	C_{BF}	0.316	0.316	....	....	....	0.414
E	FE	1.33 (2 −0.224)[c]	2.37	2.66	C_{FE}	0.250	0.251	....	....	....	0.530
	AE	1.6 (2 −0.337)	2.66	2.37	C_{AE}	0.287	0.288	....	....	....	0.470
A	BA	2.0 (2 −0.353)	3.29	1.41	C_{BA}	0.370	0.370	BA	0.467	0.454	0.299
	EA	1.6 (2 −0.287)	2.74	1.54	C_{EA}	0.337	0.343	0.513	EA	0.546	0.360
	CA	1.6 (2 −0.195)	2.89	1.49	C_{CA}	0.341	0.346	0.487	0.533	CA	0.341
C	DC	3.0 (2 −0.323)[c]	5.03	2.65	C_{DC}	0.347	0.347	....	....	....	0.345
	AC	1.6 (2 −0.341)	2.65	5.03	C_{AC}	0.195	0.195	....	....	....	0.655
D	BD	2.0 (2−0.354)	3.29	4.96	C_{BD}	0.224	0.223	....	....	....	0.601
	CD	3.0 (2−0.347)[c]	4.96	3.29	C_{CD}	0.323	0.333	....	....	....	0.399
B	FB	2.0 (2 −0.316)	3.37	1.70	C_{FB}	0.351	0.500	FB	0.514	0.492	0.335
	AB	2.0 (2 −0.370)	3.26	1.73	C_{AB}	0.349	0.500	0.521	AB	0.508	0.347
	DB	2.0 (2 −0.224)	3.55	1.66	C_{DB}	0.354	0.500	0.479	0.486	DB	0.318

[a] R/i_s. [b] $R/(R + i_s)$. [c] Estimated coefficients.

M_{CA}, will be the same. Similarly, the induced fixed-end moments for BD and DB will be the same. Furthermore, the induced fixed-end moments of the top chord members will bear a constant relationship to the fixed-end moments induced in the bottom chord members. The estimated transmission

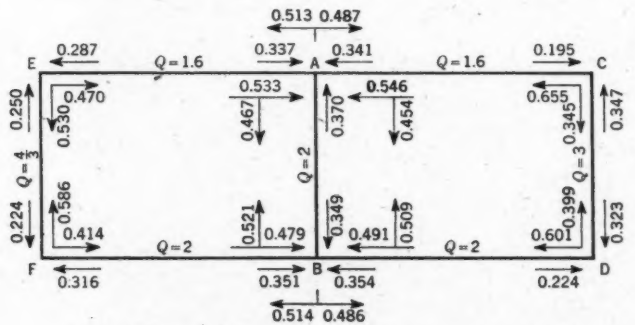


FIG. 27.—FRAME CHARACTERISTICS FOR EXAMPLE 5 (FIG. 10)

coefficients, shown in Table 20 for members EF and CD, were computed by the same method as used for Example 4. Using the transmission coefficients of Table 20, it is found that: $M_{CA} = -\dfrac{4 \times 24 \times 0.195\ (0.659 \times 3/8)}{1 - 0.0665}$ $= -4.95$ kip-ft; and $M_{AC} = -\dfrac{4 \times 24 \times 0.341\ (0.805 \times 3/8)}{1 - 0.0665} = -10.58$ kip-ft.

(a) Final Moments
(b) Sidesway Distribution

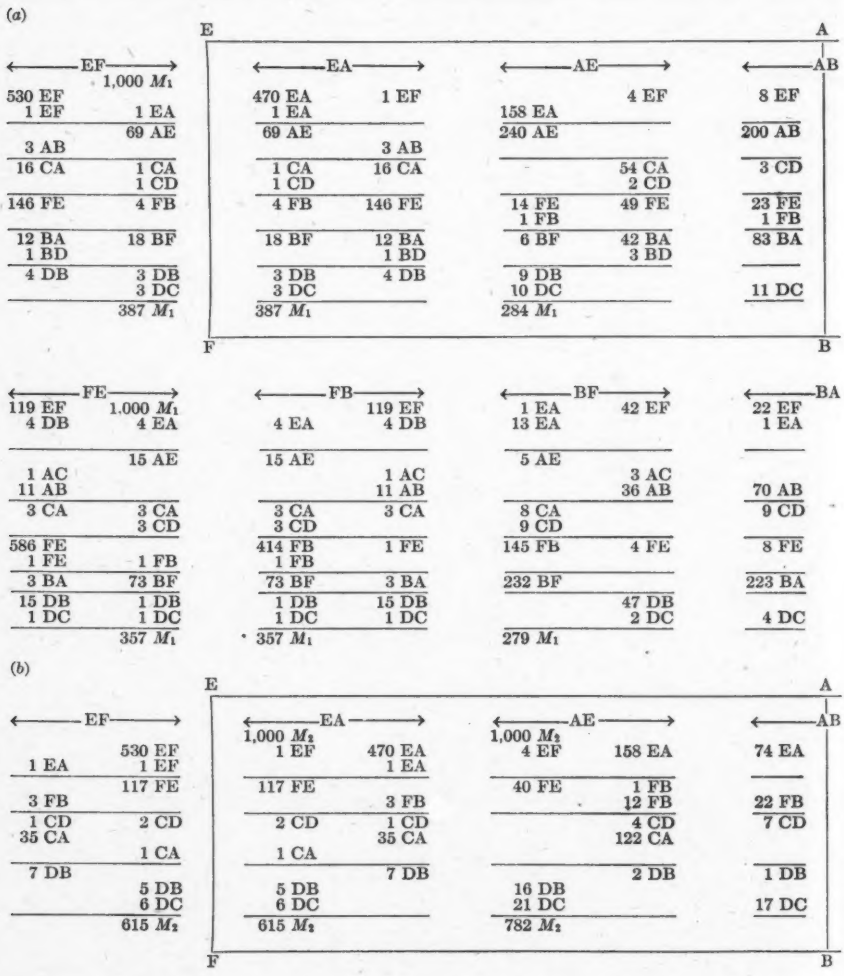
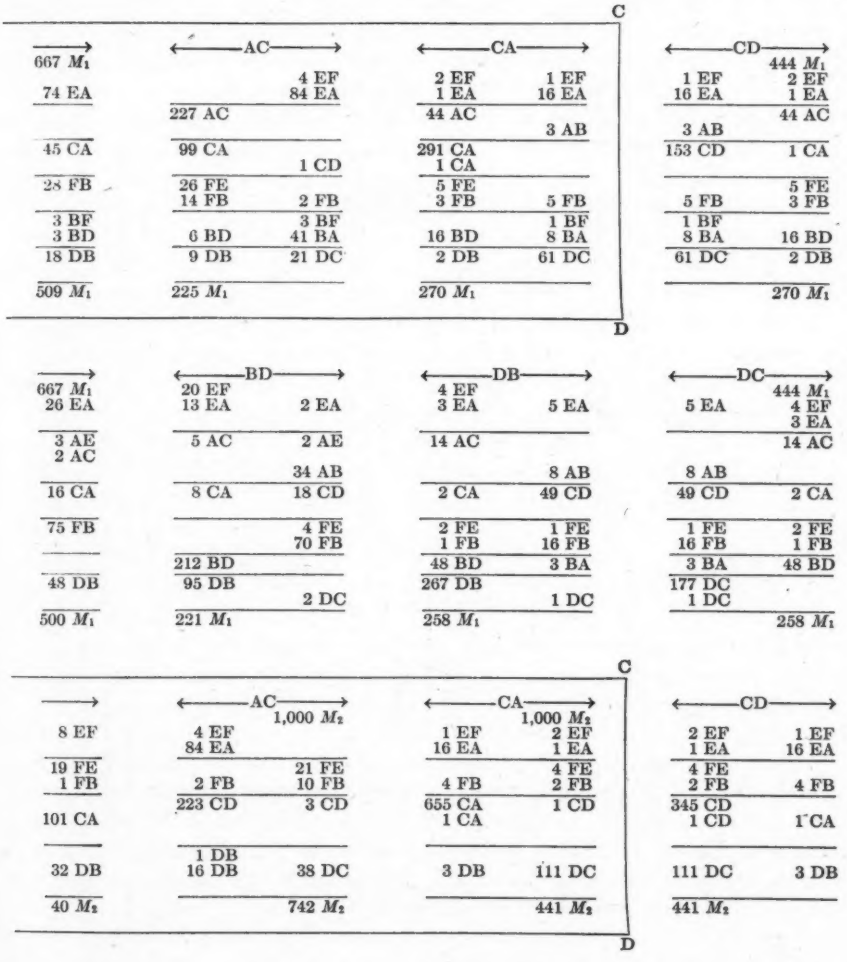


FIG. 28.—MOMENT COMPU-
(*a*) Sidesway Dis-
(*b*) Vertical Sway



TATIONS FOR EXAMPLE 5

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Distribution

| | | |

Since all horizontal members in this example are the same length (Fig. 10), the induced fixed-end moments will be proportional to the moments of inertia of the respective members, or inversely proportional to the Q-values. Therefore, if 1,000 units of moment (1,000 M_2) are distributed at each joint E, A, and C, 800 M_2 must be distributed at each joint F, B, and D. These six moments—three at 1,000 M_2 and three at 800 M_2—may be distributed concurrently, and the totals found at all joints. Since the moments at the center are equal and opposite they balance and need not be distributed. In like fashion, joints E, A, and C may be translated horizontally without joint rotation. Again, the induced fixed-end moments without joint rotation will be proportional to the moment of inertia of each vertical member. If 1,000 units of moment (1,000 M_1) are distributed at joints E and F, 667 M_1 must be distributed at joints A and B, and 444 M_1 must be distributed at joints C and D. These six moments—two at 1,000 M_1, two at 667 M_1, and two at 444 M_1—may be distributed concurrently, and the moments summed up for each joint.

The final horizontal shears in the vertical members must equal zero, and the vertical shears in the horizontal members must be consistent with the loading. Therefore, it is necessary to write the two simultaneous equations, $+ 2{,}281\ M_1 + 395\ M_2 - 1.90 = 0$; $+ 1{,}307\ M_1 + 2{,}622\ M_2 - 30.43 = 0$. When solved simultaneously, these two equations will yield values of M_1 and M_2 that will balance the shears both vertically and horizontally. By this method $1{,}000\ M_2 = 12.24$ and $1{,}000\ M_1 = -1.28$. Application of the corrections are shown in Fig. 29.

At this point, it should be noted that sidesway results from either a dissymmetry of structure, a dissymmetry of loading, or a combination of both. When vertical sway and horizontal sway can occur, simultaneously, they must be treated simultaneously. The authors have not done so in Examples 5, 6, and 7. The fact that their answers, for Example 5, are so close to the correct ones is due to the unfortunate choice of frame and loading, where dissymmetry of structure is almost balanced by the dissymmetry of loading. The fact that the sum of the authors' moments at the top and bottom of the columns does not quite add up to zero is not due to computational inaccuracies, but to the unbalancing effect of separately correcting for horizontal sway and vertical sway. When a structure has several vertical members to which sidesway may be distributed, the correction will be of little significance, but a fallacy may result, such as in this example, if such structures are used to demonstrate a principle. The question is then raised as to whether the authors' computation of sway is theoretically exact or a practical approximation.

In step (*d*) of Examples 4 and 5, the authors find the end moments of the loaded member by using the moment areas and conjugate beam, which were used in the derivation. The Brumfield method finds these moments directly from the transmission coefficients. Using the authors' carry-over factors and the Brumfield equations, the end moments for Example 4 are found to be:

$$M_{AC} = -\frac{4 \times 24 \times 0.258}{1 - 0.258 \times 0.195}\left(\tfrac{3}{8} - 0.196 \times \tfrac{3}{8}\right) = -7.80 \text{ kip-ft (authors' value}$$

7.874 kip-ft) and $M_{CA} = -\frac{4 \times 24 \times 0.195}{0.9495}(\frac{3}{8} - 0.258 \times \frac{3}{8}) = -5.48$ kip-ft) (authors' value 5.488 kip-ft). Thus, Professor Brumfield's equation is a more direct solution. In addition, his equations have been extended for uniform loads, triangular loads, and complex loads.

The authors' concept of "elastic length" is interesting but its practical usefulness is questionable. It is evident that the method could have been developed without introducing this new concept.

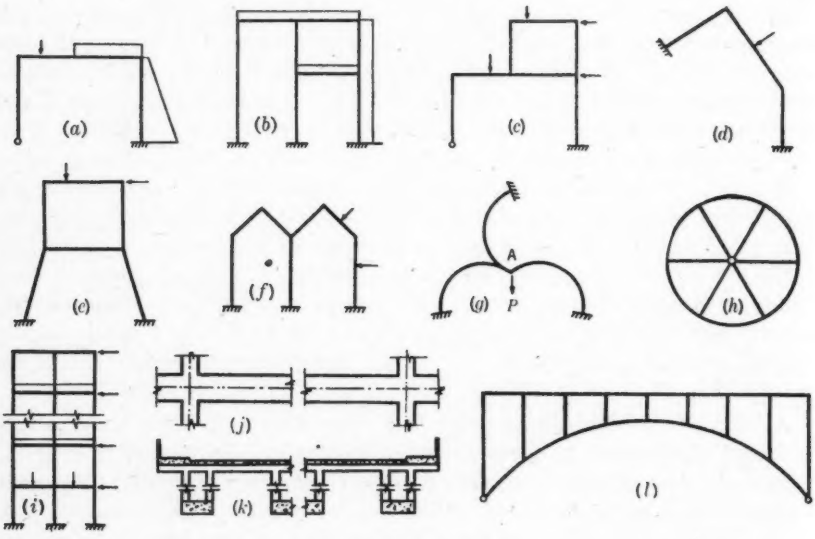

FIG. 30

Many problems confronting the practicing structural engineer are readily solved by the Brumfield method. Some of these problems cannot be solved by any other method, and it is improbable that the authors' method can be extended to be as all-inclusive. If it can be extended, however, it might be useful if the authors would indicate how the extension can be accomplished. Fig. 30 shows typical examples of some of these problems. In each of the examples shown the moment of inertia may be a variable with little additional work.

Fig. 30 (*l*) shows an arch rib with spandrel columns and deck. The analysis of this structure is complicated by the fact that the deck, the spandrel columns, and the ribs will participate in supporting the loads and in resisting temperature stresses. The problem is so complex that, ordinarily, bridge designers will avoid the analysis by judicious use of expansion joints in the deck structure. This procedure obviates the beneficial effects that the rib derives from deck participation.

The Brumfield method uniquely solves the problems presented. It is universal in its application in that it develops transmission coefficients for curved members, as well as for straight members and for members with variable moment of inertia. These transmission coefficients take into account the actual restraints at the ends of the member. If the connection of a member yields and the amount of yielding is known, it may be incorporated into the analysis with little difficulty.

The outstanding feature of all methods of direct moment distribution is that the moments "flow away" from the point of origin in one direction—never returning to that point except in re-entrant frames. The structural "keenness" developed by use of direct methods far exceeds that developed by methods employing successive approximation. This in itself is a strong argument for wide adoption of direct methods such as those proposed by the authors and discussed herein.

Correction for *Transactions:* In November, 1947, *Proceedings*, pages 1419 and 1422, Figs. 14 and 16 should be transposed.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

APPLICATION OF GEOLOGY TO TUNNELING PROBLEMS

Discussion

BY BERLEN C. MONEYMAKER, AND ROBERT S. MAYO

BERLEN C. MONEYMAKER,[2] ASSOC. M. ASCE.—The application of geology to problems in tunneling, as presented in this paper, is interesting and timely. Mr. Wahlstrom places at the disposal of the engineer and the inexperienced geologist a concise and well-written account of the manner in which certain geologic conditions affect tunneling. The author not only presents a very clear, rather elementary discussion of the characteristics of the more common rocks and their primary and secondary structures, but also indicates several geologic conditions that may affect tunneling adversely and suggests ways of eliminating or reducing the resultant difficulties.

The writer has made both preliminary and final geologic studies of many miles of tunnels, both within and without the United States. Although no issue is taken with either the author's statements (under the heading, "The Functions of the Geologist"), or the facts he states regarding rock characters and rock structures, the paper tends to convey the idea that the geologist has more to do with planning a tunnel than is actually the case. Generally, he has little to do with determining the alinement, and even less to do with determining the size of the bore and the shape of its cross section, especially if the tunnel is for the diversion of water for power, irrigation, or water supplies. In this case the engineer determines the points to be connected by a tunnel and the amount of water to be carried and then calculates the size and the shape of the cross section of the bore which best meet the requirements. In the case of highway and railroad tunnels, the engineer also determines the alinement, the size, and the shape of the cross section. The location of such proposed tunnels may be surveyed and staked before the geologist is asked to make his studies. The geologist may or may not have more to do with the planning of tunnels in mining districts, where the tunnels are for exploration, mine development, or drainage.

NOTE.—This paper by Ernest E. Wahlstrom appeared in October, 1947, *Proceedings*.

[2] Chf. Geologist, TVA, Knoxville, Tenn.

In 1937 the writer was asked to make a geologic study of the Garzas tunnel in Puerto Rico. This tunnel, for the diversion of water from the Garzas reservoir for power and irrigation, was already beyond the planning stage and had been surveyed. It is 11,640 ft long and extends in a north-south direction across the rugged Cordillera Central, which forms the insular divide. The ground to be penetrated had not been explored by core borings, test pits, trenches, or any other method and, apparently, no funds were available for exploration. As most of the tunnel was to be from 400 ft to 1,200 ft below the surface, sufficient exploration to yield detailed information was not feasible. There was a good exposure of bedrock at the intake portal and small outcrops in the beds of two of the several small streams crossed by the tunnel line. Elsewhere bedrock was concealed by as much as 200 ft of residuum. The entire tunnel location was covered by a dense jungle of tropical vegetation, except along the cleared parts of the survey line. Under such conditions the geologist is at an acute disadvantage. By a very careful study of the stratigraphy and the orogenic and geomorphic history of the region as a whole, the writer was able to predict accurately the nature of the geologic problems to be encountered. A painstaking study of the residuum along the survey line revealed the character of the underlying rock with a very high degree of accuracy, and a study of the only outcrops on the line revealed the location of a few faults. Although it was believed that hundreds of faults would be crossed by the tunnel, there was no method of determining their location, their magnitude, or their seriousness as tunnel problems.

The tunnel, a 7-ft horseshoe in cross section, was driven from the two portals and was "holed-through" near the midway point. As predicted, the northern 10,486 ft of the bore penetrated water-laid volcanic ash shales and associated limestones; the remaining 1,188-ft section penetrated massive tuff and agglomerate of andesitic composition—both types of rock being cut by numerous intrusions of andesite porphyry. Hundreds of faults were encountered, but most of them presented no serious problems. Many involved sizable zones of closely fractured rock and resulted in considerable overbreakage. These structures required timbering during construction and lining afterward. Two major faults, both apparently active, were crossed with some difficulty and after considerable delay. Both yielded copious quantities of water under considerable head. One resulted in the caving of the heading and the partial filling of the bore with 1,000 cu yd of fine gouge and 500 cu yd of broken rock. This fault zone was 152 ft wide and yielded 1,300 gal of water per min. The other fault also resulted in the failure of the heading and the partial filling of about 40 ft of the bore with broken rock and gouge. The fault zone was only 21 ft wide and yielded 200 gal of water per min under a head of 160 lb per sq in.

The location of the spillway tunnel at Garzas dam was also examined by the writer before its construction was undertaken, but after it was beyond the planning stage and after it had been surveyed. This tunnel, 1,100 ft long, was designed as a large bore of horseshoe cross section. Bedrock was exposed at the outlet portal and near the intake, but it was concealed elsewhere by a thick residuum. The ground to be penetrated was explored inadequately by a few drill holes. Geologic studies, as detailed as could be made under the

circumstances, afforded no indication of serious problems, but, as the construction neared completion, a fault zone was encountered and occasioned a long delay and considerable difficulty.

The Apalachia tunnel, constructed by the Tennessee Valley Authority to divert water from the Apalachia reservoir to the powerhouse 12.6 miles downstream, is located on the south side of Hiwassee River in Polk County, Tennessee. Although the terrain to be crossed by this tunnel was thoroughly studied geologically and mapped on a scale of 1 in. to 500 ft before the project reached the planning or designing stage, geologic conditions played little or no part in its alinement and design. It is 43,637 ft long, and was driven as a bore of circular cross section, from 20 ft to 22 ft in diameter. It crosses the strike of the Great Smoky (Thunderhead), Pigeon, Citico, and Wilhite formations, which consist of altered fine to coarse clastic sediments. The geologic structure is moderately complex and is characterized mainly by folds and thrust faults. Exposures of bedrock were sufficiently large, numerous, and continuous in the vicinity of the tunnel alinement to permit accurate projection to the tunnel grade of all formation contacts and changes in lithology and in the larger structural features. Although numerous faults were encountered, no difficulty of any kind was experienced in driving the tunnel.

Another tunnel constructed by the Tennessee Valley Authority in the highly deformed metamorphic rocks of the Southern Applachians is the Ocoee No. 3 tunnel, which diverts water from the Ocoee No. 3 reservoir to the powerhouse 4.2 miles downstream. This 13,000-ft-long tunnel was driven as a bore of horseshoe cross section. From a geologic study of the surface along the survey line, all the larger geologic structures and changes in lithology were projected accurately to the tunnel grade. Even though the geologic structure was characterized by numerous folds, faults, and joints, no problems were encountered. There were a small amount of weathered rock and a few zones of shattered rock which required support, but these conditions presented no difficulties.

In all previously mentioned tunnels, and numerous others on which the writer has done geologic work, it would not have been feasible to follow some of the author's recommendations. First, the alinements were determined by engineering and other requirements. Moreover, the direction of strike and dip of the strata and foliation changed so many times in each of the tunnels that it would have been impossible to keep the bore advancing normal to such structures and as stated (under the heading, "Physical Characteristics of Consolidated Rocks") "* * * so that the layers will dip away from the headings." The rocks penetrated by these tunnels are cut by numerous joints and in most cases the most prominent joints are dip joints, which are parallel to the dip and normal to the strike. Where the most prominent joints are dip joints, a tunnel normal to the layers will be parallel to the joints That no tunnel could be driven normal to two structures which intersect at right angles is obvious. Furthermore, in spite of the advantages of altering the cross section of a tunnel, as Mr. Wahlstrom suggests (under the heading, "The Tunnel Cross Section") "in response to changes in the rocks and rock structures," it would not be

practicable to do so in the case of tunnels where hydraulic considerations are important.

In large diameter tunnels, especially those constructed for use in hydroelectric projects, such features as bedding planes, flow lineation, foliation, joints, and minor faults are of relatively minor importance. Large faults and fault zones of badly weathered and weak rock, swelling ground, large flows of water under high head, and high temperatures are much more serious. In a great majority of cases, some of these conditions cannot be accurately predicted, even by experienced tunnel geologists.

In pointing out the difficulty of locating and designing tunnels so as to take full advantage of geologic conditions, the writer does not question the validity of the author's conclusions. Rock conditions and rock structures do affect tunneling precisely as he states. Although, generally, it is not possible to meet all engineering and other requirements and at the same time to locate and design the tunnel to the best advantage geologically, an experienced tunnel geologist can undoubtedly contribute substantially to the economy of funds and of time in the planning, design, and construction stages.

ROBERT S. MAYO,[3] M. ASCE.—The field that the geologist should occupy during the planning and construction phases of tunnels is clearly indicated by this excellent paper. Professor Wahlstrom's experience has apparently been gained in the deep mines of the west in connection with the driving of haulage and drainage tunnels. In this type of work, where the tunneling is generally done by company forces, the advice of a trained geologist in selecting the route for the safest and most economical tunnel is of utmost importance.

To the tunnel engineer on the eastern seacoast of the United States, such opportunities for profiting by the advice of the geologist are few. The tunnel route is fixed in most cases—the railroad engineer has his portals all staked out for him, regardless of the intervening ground; in the larger cities, the sewer engineer must run an intercepting tunnel down the center of certain streets from trunk sewers to the sewage treatment plant. Only in the rare case of a diversion or aqueduct tunnel in an unsettled area can the tunnel be shifted an appreciable amount to secure better ground.

This does not mean that the engineer cannot use the services of the geologist. The specifications prepared for the information of bidders would be of much greater value if there were a section in which a trained geologist analyzed the borings and other available information in order to forecast the types of ground to be found, the possibility of encountering faults, and how the rock will break; but this introduces an objection. The engineer has found, by sad experience, that it is rather expensive to hazard a guess to the contractor as to what type of ground he will encounter. In presenting the log of borings in the bidding documents, most engineers specifically state that the findings shown in the log are not guaranteed and that the contractor must make his own interpretation of the borings. To incorporate in the specifications a geologic monograph stating what types of ground are likely to be encountered, what difficulties might be expected, and what overbreak to anticipate, would result

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in a lower bid by the contractors in most cases. The courts have been extremely liberal in interpreting implied geologic data in the contractor's favor and, therefore, many engineers would be hesitant to include such forecasts in the specifications, when it might implicate the owners in expensive lawsuits, if the geologist's opinion were wrong.

This an unfortunate situation, but it does not lessen the value of geologic advice during the planning and bidding stages of a job. Consultation with the geologist would certainly enable the designing engineer to establish the "A" and "B" lines of the tunnel more intelligently. These lines govern the thickness of the concrete lining, and at present they are established by "rule of thumb."

The writer has stated that, in the past, the routes of tunnels are selected by factors other than geological ones; but, nevertheless, should the geologist indicate that extremely bad ground is to be expected, in most cases the engineer would call for a re-examination of the entire project with major relocation of the tunnels to secure better ground.

In spite of the present legal pitfalls that make it unwise for the engineer to add to the bidding specification the opinion of a trained geologist, the geologist can be of great value to the owner's engineer. By reading the daily reports of earlier tunnels, he can deduce the type of rock penetrated. He can forecast, with fair accuracy, the results of the drilling, the type of ground to be penetrated, and the difficulties (particularly "swelling ground") to be anticipated. He can be of real help in setting the "A" and "B" lines, in addition to recommending whether grouting should be specified and just what quantity of grout each type of ground will take.

The geologist can be of immense value on all tunnel projects, not only to the engineer, but to the contractor. To the end that every job can benefit from geologic science, it is recommended that, when an organization is not large enough to warrant a full-time geologist, that a consulting geologist be engaged on a part-time basis to visit the tunnel at least weekly to report on the ground and its effect on progress and overbreak. Complete engineering and geological reports on every tunnel job will be of immense value to the next generation.

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DISCUSSIONS

CLASSIFICATION AND IDENTIFICATION OF SOILS

Discussion

BY KENNETH S. LANE, GEORGE F. SOWERS, RENÉ S. PULIDO Y MORALES, RAYMOND F. DAWSON, AND D. F. GLYNN

KENNETH S. LANE,[9] ASSOC. M. ASCE.—Of the eight classification systems presented in the paper, the writer has had occasion, since 1937, to work with four—the triangle textural classification, the master curve textural classification, the Public Roads Classification, and the Casagrande system. From this experience the writer can certainly agree with the author's designation of soil classification as the most confused chapter in soil mechanics. Although the author modestly terms the Casagrande system as the Airfield Classification System, the writer prefers to designate it as the Casagrande classification, since it has been widely known as such from its initial publication in 1942. This classification is easily the best of the aforementioned four systems (in their forms as described in the paper) for the airport-highway type of work for which it was originally designed. In its application to dams and levees, it has certain limitations which can be best considered from their relations to the primary requirements of a soil classification system intended for wide usage.

Requirements of a Classification System.—The requirements particularly considered are those for a large and widespread organization handling many projects of all sizes; for less intensive operations, or for work concentrated in a particular region, simpler requirements might suffice. In this type of work, boring logs generally include a classification of the soil into a group or type, plus a word description similar to the descriptive classification covered in the paper (under the heading, "15. Descriptive Soil Classification").

On a large airport, levee, dam, or canal, involving a hundred or more foundation explorations, it is generally not feasible to consult the original boring logs whenever questions arise, so these are usually filed as reference material.

NOTE.—This paper by Arthur Casagrande appeared in June, 1947, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1947, by Ralph E. Fadum; October, 1947, by James H. Stratton, and Donald J. Belcher; and November, 1947, by J. A. Haine and J. W. Hilf, and Jacob Feld.

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Rather, the usefulness of a soil classification system comes from its application to graphic abstracts of boring logs, wherein the most pertinent information is abstracted and placed on boring drawings. Hence, for a large project, this graphic abstract is the form of boring information most used by engineers in designing, by bidders in estimating, and by contractors and field inspectors in planning construction. For this purpose, it is essential that the classification system include some type of shorthand notation to convey maximum information at a glance. All engineering classifications described by the author are so designed, except for one variation of the textural system where the words "some, little, and trace" indicate decreasing percentages of size fractions, and require slow reading for understanding.

For this type of work, the writer feels that a soil classification system should satisfy the following four requirements:

Requirement A.—It should describe the soil in well-understood terms, which convey an idea of its type and behavior.

1. In addition to soil specialists, the terms used should convey the intended meaning to designing engineers, inspectors, geologists, contractors, and courts of law, which last are the recipients of altogether too many disputes, involving boring information versus foundation conditions encountered.

2. The system should make allowance for regional use of certain terms, particularly meaningful in that region, such as glacial till, buckshot clay, caliche, and others.

Requirement B.—The system should furnish an indication of soil properties and performance.

1. It should permit a crude estimate of permeability, compressibility, and shear strength—the three primary properties of a soil. When combined with other boring information (compactness and water table location), the classification should permit a rough estimate of soil performance, such as frost susceptibility, drainage, settlement, slope stability, wheel load capacity, and compactibility (probable range of compacted density and normally suitable compaction procedure).

2. It should be capable of subdivision. Soil is so complex that no classification system can replace tests. Nevertheless, subdivision can often reduce testing by narrowing the range of properties for soils, at least within a particular region. Approximate correlations of soil properties and performance experiences with the indicators of a classification system often can be facilitated by subdivision, particularly when the testing program extends over a length of time sufficient to accumulate experience from past tests. Furthermore, such correlations, even though rough, are often a "must" for rapid handling of a large number of projects whereas necessity frequently requires that testing be limited, or omitted entirely, for smaller projects.

Requirement C.—It should be applicable from visual examination—both in a simplified form and, with experience, in a more refined form.

1. The simplified form should be suitable for determination by boring inspectors from visual examination.

2. The refined form should be such that: (*a*) It can be assigned by a well-trained classifier or chief boring inspector from visual examination, supplemented by occasional check tests; or (*b*) it can be determined from tests in the case of a less experienced classifier. For the latter case, and also so that different laboratories will arrive at the same result, reasonably definite numerical limits are necessary.

Requirement D.—It should employ a simple system of notation for graphic abstracts of boring logs on drawings recording boring information.

1. The notation should register rapidly in the mind while the reader is scanning and comparing plots of several different borings.
2. The drafting effort should be minimized in the preparation of boring drawings, including those suitable for reduction. These are generally of two types: (*a*) Contract drawings to serve bidders, contractors, and construction engineers, and which also usually form one group of exhibits in any court case; and (*b*) study drawings (usually foundation profiles) to serve soils engineers, geologists, and designing engineers in developing the design treatment and in illustrating its adequacy in reports. Unlike study drawings, contract drawings should be made to avoid interpretation, from the legal standpoint that information furnished to bidders should be basic data—not interpretations.

Requirement A, for naming a soil to cover its type and behavior, has been complicated by the practice of using the soil names, "gravel, sand, silt, and clay," to represent, also, grain-size fractions as arbitrarily delineated by one of the grain-size scales in Fig. 2. The terms, "gravel, sand, silt, and clay," describe common soil types, and like all natural soils, each is usually a composite mixture of from two to four of the size fractions designated as "gravel, sand, silt, and clay sizes." Good definitions of these four soil types have been given by H. A. Mohr, M. ASCE (23),[9a] and also in dictionaries; and the terms convey about the same meaning to engineers, geologists, contractors, and courts. The general behavior of each of these four soil types was fairly uniformly understood until soil technicians began to confuse themselves and others by using the same words for size-fraction names. This practice has degenerated to the extent that some laboratories assign a name to a soil from the percentages of different size fractions, according to certain charts or classification triangles like those in Fig. 3; sometimes this is done as a routine matter from a grain-size curve by a classifier who never even sees the sample.

If the size-fraction percentages were a reliable measure of soil behavior, this practice might be condoned; but such is far from the case, as the author has indicated, mentioning the common illustration of plastic lean clay and fine cohesionless silt or rock flour of the same gradation. As one example, samples of Manhattan silt having 20% in the clay-size fraction would be designated in some classifications as silty or lean clay. Since this material becomes quick on the slightest provocation, it would indeed be unfortunate for any contractor (and later for the owner in court) who relied on the cohesion of a true lean clay in attempting to excavate below ground water.

[9a] Numerals in parentheses, thus: (23), refer to corresponding items in the Bibliography (see Appendix of the paper), and at the end of discussion in this issue.

Although frequently of the same gradation, classification of loess as silt ignores the fact (and the consequence) that loess often undergoes a serious rapid settlement from collapse of structure under the influence of seepage, whereas silt is seldom so affected. Because of the angularity of its coarse particles and its greater compactness, glacial till generally has greater shear strength, lower permeability, and higher resistance to excavation than a rounded particle gravel of equal gradation. These and other misleading results obtained with textural classifications have convinced the writer of the necessity for requirement A in naming the soil to describe its type and behavior.

Casagrande Classification.—In most usage to date, the Casagrande classification has generally been applied in the form of the group symbols GW to Pt, as shown in Table 4, Col. 3. When so applied, it is actually a plasticity classification for fine-grained soils, and a textural classification for coarse-grained soils. Since it is based on the position and shape of the entire gradation curve, the textural classification phase is superior to the triangle classification, which is based on only three points of the gradation curve. The shape and position of the entire gradation curve have considerable bearing on important soil properties. This fact is widely recognized and is illustrated by many past attempts at better definition—such as the effective size and uniformity coefficient presented by Allen Hazen; the mean diameter and grade line introduced by Frank B. Campbell, M. ASCE (8); and the mean size, fineness dispersion, and curve shape proposed by D. M. Burmister, Assoc. M. ASCE (46)(47).

Hence, although this textural phase of the Casagrande classification is one of the best of the systems described in the paper, it is subject to the general objections to a textural classification when used alone. For example, such unlike materials as gravel, cohesionless till, talus, and caliche would be classed in group GW, and silt and loess in group ML. From the closing paragraphs of the paper, it is apparent that the author is aware of this deficiency (or possible misapplication) of the system and intends that the group symbol should be supplemented with a soil name. The writer is in agreement with this form of the system, as it then meets requirement A.

For furnishing indicators of soil properties (requirement B), the writer considers the Casagrande classification the best of any yet devised in regard to compressibility and shear strength of fine-grained soils. However, its performance has been relatively poor in indicating the permeability of coarse-grained soils. The writer has experienced ranges in permeability in ratios of from 1 to 10,000 and even from 1 to 100,000 which are considered undesirably wide for a single-class group. (Although many past attempts have shown uncertain relationships between permeability and results of simpler classification tests, the lack of success in deriving these relationships is probably explained by the difficulties encountered in permeability test procedure. With more refined testing techniques, considerable data have been accumulated since 1942 (unpublished) which, it is believed, indicate that such relationships do exist; and, of the simpler classification systems available at present, the data correlate best with a master curve system.)

If subdivisions were feasible, they would probably narrow this permeability range, as well as the range in compacted density, which has also been experi-

enced as relatively widespread in certain groups. After several unsatisfactory attempts at subdivision, the writer has joined in the frequently-voiced comment that the Casagrande classification is not easily adapted to subdivision. In discussing possible expansion of the system in Section 14 of the paper, the author offers a scheme of subdivision, partly by the addition of new groups (GU, SU, KL, KI, and KH) and partly by refinement using boundary groups (GW–SW, GC–SC, and so on) and by use of subscripts m, c, u, w, and p. When subdivision is practiced, it is preferably accomplished by refinement, rather than by addition, because the latter makes less feasible any later attempts at correlating data gathered prior to expanding a classification system by addition. The proposed expansion of the Casagrande classification to represent a soil as $GF_m - S_wF_c$ (to give, possibly, an extreme case with the binder near the boundary between plastic and nonplastic) would seem to indicate (by its complexity) the difficulties of subdivision.

In respect to requirement C, for determination by visual examination, the Casagrande classification is one of the best, and the author's methods for field identification of fine-grained soils in Section 13 of the paper are worthy of widespread application. However, for checking between different laboratories, it is desirable to have numerical limits for the results of usual classification tests, without the necessity of transmitting a sample for examination. Such limits are also of considerable value for checking classification of a soil by tests, especially in the case of an inexperienced classifier. For fine-grained soils, the Casagrande classification provides such limits, but only to a minor degree for coarse-grained soils.

A good feature of the Casagrande classification is the use of a simple system of notation for boring drawings (requirement D), unless it is complicated by the aforementioned scheme of subdivision. Possibly the best of the simpler systems yet devised for registering quickly in the mind is the system of rectangular and diagonal hatching usually applied with the triangle classification in Fig. 3(*b*), in which heavy symbols indicate the predominant fractions, and lighter symbols denote the two minor fractions of the three main grain-size fractions (48). However, the drafting cost required for suitable reduction is so much greater for this system of hatching (generally requiring inked tracings) that it more than offsets the advantage of quick registering.

QPC Classification.—These difficulties experienced in applying the Casagrande classification, and the greater limitations of the other systems discussed by the author, have led the writer to seek a combination of the better features of several systems. Since 1943, the result has gradually evolved into a combination of soil names from the descriptive classification; a gradation identification from the master curve system started on Quabbin Dam in Massachusetts by Karl R. Kennison and Stanley M. Dore, Members, ASCE (7), and expanded by Frank E. Fahlquist, M. ASCE, and Waldo I. Kenerson into the Providence classification (8); and the plasticity characteristics of the Casagrande classification. The combination, in these three parts, has been tentatively termed the QPC classification after its principal sources (Quabbin, Providence, and Casagrande).

Part 1. Name of Soil Type.—This consists of a soil name, generally with one modifying adjective, selected primarily to describe soil type and behavior rather than gradation. It is chosen from visual examination (preferably on a moist sample) to agree with local semitechnical usage by engineers, geologists, contractors, and courts.

Examples of Soil Names.—The following are typical examples of names, used as applicable to local conditions, with additions as necessary.

Material	Description
Gravel	Variably graded combination of pebbles and sand, sometimes including some silt and clay. Coarser particles, from 6 in. to ¼ in., at least partly rounded from fluvial action.
Sand	Uniformly graded, bulk of material smaller than ¼ in., individual particles readily visible to the naked eye.
Silt	Fine grained, lacking plasticity, little or no cohesion when dried, majority of individual grains not visible to the naked eye. Shaking test shows quick or "livery" condition.
Clay	Fine grained, plastic, and cohesive, strong when dried.
Till	Variably graded, unsorted glacial deposit, characterized by angularity of coarser fragments, generally very compact. Indicate whether cohesive or cohesionless (dependent on character of fines).
Loess	Unstratified aeolian deposit, homogeneous powdery soil of silt sizes, structure loose and inclined toward vertical cleavage.
Varved Clay	Alternating thin beds of clay and silt, interstratified, clay-like behavior.
Gumbo	Highly plastic clay, extremely sticky when wet, often a product of weathered shale (localized term).
Talus	Variably sized angular rock fragments, fallen to base of slope, voids seldom all filled.
Decomposed Rock	Structure preserved in place, but chemically disintegrated to soil sizes (with bonding largely destroyed). Also include name of rock.
Coral Sand	Ground-up coral.
Blow Sand (Dune Sand)	Wind-borne, free-moving sand, particles at least partly rounded.
Buckshot Clay	Compact clay, persistently breaking into uniformly small-sized fragments (localized term).
Diatomaceous Earth	Siliceous soil composed chiefly of skeletons of minute organisms.
Topsoil	Weathered surface soil containing organic matter, usually supporting vegetation. Termed loam in some regions.

Examples of Stratum Names Where Grain-Size Number Not Applicable.—

Material	Description
Peat	Partly carbonized vegetable matter, odoriferous, and fibrous. Swamp deposit.
Fill	A man-made deposit. Describe constituents, as cinder fill, rubbish fill, sawdust, ashes, cans, bricks, broken concrete, and so on. Grain-size number used only when soil a major constituent, or when knowledge of gradation desirable, as with slag, cinders, or earth fill.

Examples of Modifying Adjectives.—As frequently used on graphic abstracts of boring logs:

Coarse	Gravelly	Organic	Kaolinitic
Medium	Sandy	Micaceous	Bentonitic
Fine	Silty	Cemented	Carbonaceous
	Clayey		Calcareous

Fat, Lean } Applied to Clays

Generally not more than one adjective is used to supplement the soil name on graphic abstract logs and it is omitted when not particularly significant.

The system is flexible in allowing for the addition of terms of particular regional significance and certain geologic terms well characterizing soil behavior. Devices for obtaining a soil name, such as a classification triangle, have been purposely omitted in order to avoid classification by rote, and its resultant misleading information. For selection of the soil name, guidance of geologic training is helpful; and, for an inexperienced classifier, it might be advisable to add a few remarks to the foregoing definitions to cover behavior similar to those given by H. A. Mohr (23).

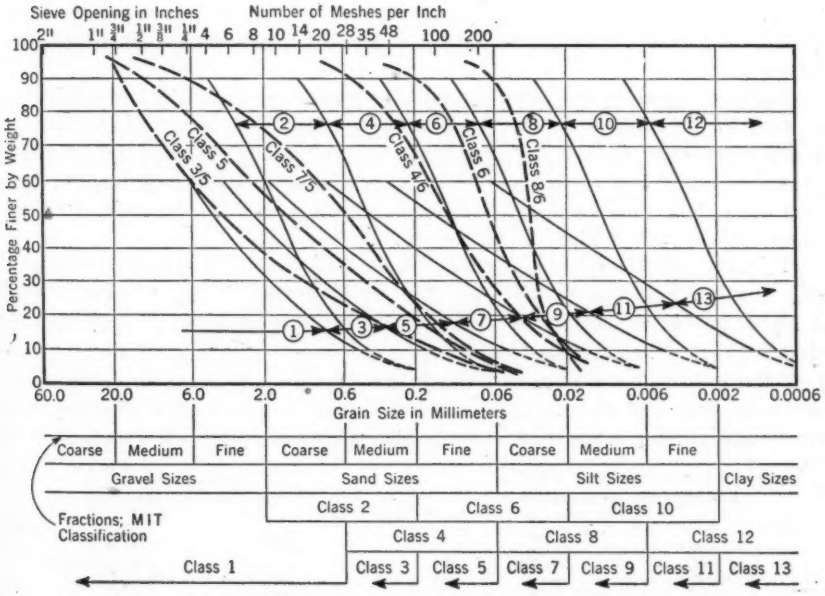


FIG. 8.—LIMITS OF GRAIN-SIZE NUMBER CLASSES

Part 2A. Grain-Size Number.—Part 2A consists of a grain-size number representing position and shape of the entire gradation curve. Thirteen basic classes are recognized as defined by the master curve bands in Fig. 8. Even numbers indicate predominantly uniform gradation; odd numbers indicate predominantly variable gradation.

When a word description is advisable for those unaccustomed to thinking in terms of gradation curves, the thirteen basic classes are defined as follows:

Class	Description of Gradation
1	Graded from gravel to coarse sand sizes with little medium sand.
2	Coarse to medium sand sizes with little gravel and fine sand.
3	Graded from gravel to medium sand sizes with little fine sand.
4	Medium to fine sand sizes with little coarse sand and coarse silt.
5	Graded from gravel to fine-sand sizes with little coarse silt.
6	Fine sand to coarse silt sizes with little medium sand and medium silt.
7	Graded from gravel to coarse silt sizes with little medium silt.
8	Coarse to medium silt sizes with little fine sand and fine silt.
9	Graded from gravel or coarse sand to medium silt sizes with little fine silt.
10	Medium to fine silt sizes with little coarse silt and clay.
11	Graded from gravel or coarse sand to fine silt sizes with little clay.
12	Fine silt to clay sizes with little medium silt and fine clay.
13	Graded from coarse sand to clay sizes with little fine clay.

In these descriptions the terms designate size fractions of the M.I.T. classification. A much better system would be the eventual acceptance of other terms for the size fractions which would not conflict with "gravel," "sand," "silt," and "clay," as soil types. Names from a dead language such as Latin, or the use of Greek letters might be acceptable, or possibly terms such as "coarse," "fines," "superfines," and "colloida." However, any possible terms would need careful consideration for correlation with languages other than English.

Samples are classified visually, using reference jars of materials graded according to the thirteen basic classes, supplemented by occasional check tests. When determined from a test, the grain-size number is obtained with a transparent celluloid reproduction of Fig. 8, placed over the gradation curve in question. For distinguishing visual classifications, these are followed by the letter "v," as "4 v," "7/9 v," and so on.

Primary subdivision is accomplished by double class numbers to represent soils grading from one band to another as illustrated in Fig. 8, thus: "8/6." A higher number over a lower number indicates a more uniform gradation (as class 8/6) and a lower over a higher number a more variable gradation (as class 4/6) than that of the standard band (as class 6). In forming double class numbers, even and odd numbers are never combined. Double class numbers are recognized for a spread of two bands, which is quite common, and is as follows:

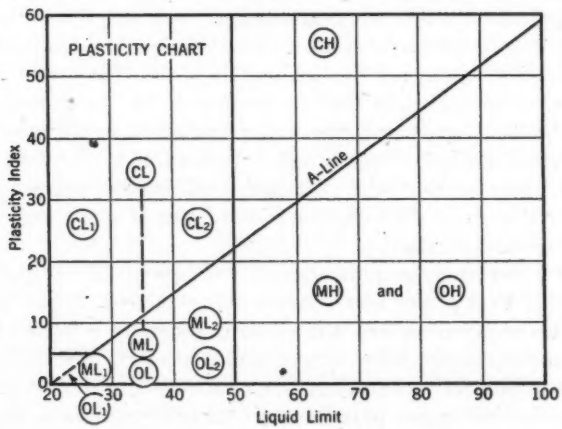
Basically variable gradation		Basically uniform gradation	
1/3	3/1		
3/5	5/3	2/4	4/2
5/7	7/5	4/6	6/4
7/9	9/7	6/8	8/6
9/11	11/9	8/10	10/8
11/13	13/11	10/12	12/10

With this primary subdivision, the thirteen basic classes are expanded to thirty-five grain-size number classes which have been found to cover most needs of the several hundred thousand samples to which this system has been applied.

Rare cases of extremely variable soils grading over a wide range can be handled by recognizing a spread of three bands from lower to higher odd numbers: 3/7, 5/9, 7/11, and so on. However, it is not advisable to reverse this by recognizing a spread in odd numbers of three bands from higher to lower numbers, since the result is one of the uniform basic classes. Thus gradations would not be designated as 7/3 or 11/7; rather these would be noted as classes 2 and 4, respectively.

If further refinement is desired, it may be accomplished by a secondary subdivision to recognize border classes, although the writer has seldom considered this necessary. A border gradation may be defined as one lying near the boundary of two of the adjacent master bands in Fig. 8. Such a gradation is represented as class 4–6 to indicate gradation near the boundary between classes 4 and 6. Other than for attempting close refinement, border classes allow for cases of uncertainty in visual classification, as class 4–6 v.

TABLE 6.—PLASTICITY-CHARACTERISTICS SYMBOLS FOR FINE-GRAINED SOILS WITH LITTLE OR NO COARSE AGGREGATE



Low plasticity	L	M	Behavior characteristics of silt
		C	Behavior characteristics of lean clay
When subdivided:		O	Organic content causing plasticity
L_1 = slight plasticity			
L_2 = moderate plasticity			
High plasticity	H	M	Behavior characteristics of quite plastic silts, such as micaceous, diatomaceous, or elastic silts
		C	Behavior characteristics of fat clay
		O	Organic content causing high plasticity, such as organic clays

In such cases of uncertainty, the last number, as "6 v," is chosen as the more probable one. Then, for simplicity, combined class numbers (whether 4/6, 8/6, or 4–6) can always be considered from the standpoint that the last number (in this case, class 6) approximates the gradation—and conservatively

so, because it indicates the range of finer sizes. Hence, the thirteen basic classes are the ones to be fixed in the mind. Once these are understood, combined class numbers are easily visualized as variations.

Part 2B. Plasticity Characteristics.—This phase consists of plasticity characteristics symbols taken from the Casagrande system and, likewise, applicable only to fine-grained soils with little or no coarse aggregate. The proper symbol is determined visually as described by the author; or, where Atterberg limit tests are conducted, it is determined from the plasticity chart in Fig. 4. In Table 6 this chart has been redrawn to follow the author's suggestion of dividing the L-group at a liquid limit of 35. However, the writer prefers to retain the original L-group as the simple form (requirement C) and to use the two subdivisions as the refined form designated by subscripts as shown in Table 6.

In application of the QPC system, classification is accomplished visually by examining moist samples—for fine-grained soils, employing the shaking test and examination of plasticity characteristics as the author describes (under the heading, "13. Field Identification of Fine-Grained Soils"). Laboratory tests are employed only when the classifier feels uncertain, or for purposes of checking, or for training an inexperienced classifier.

Boring logs include soil name, grain-size number, and, for a fine-grained soil, the plasticity-characteristics symbol. Also included is a complete description covering color, particle angularity, consistency, plasticity, and dry strength. Such description is similar to that in Section 15 of the paper, except that, considering the legal viewpoint, the writer prefers to avoid interpretation by using adjectives describing compactness. Instead, it is preferred to give basic data without interpretation wherever such basic data are available, as blows per foot required to drive the sampling spoon (sizes of spoon and hammer, and height of fall being indicated).

On contract drawings, graphic abstracts of logs include the symbols aforementioned under Part 2, plus a brief name of the soil from Part 1 for the benefit of bidders. On study drawings, the name is generally included only when it would add some significant information to that already indicated by the grain-size number and plasticity-characteristics symbol. The simpler form (requirement C) includes soil names plus the thirteen basic grain-size number classes with only limited use of border classes. The refined form includes soil names plus grain-size numbers from the thirteen basic classes and from the primary subdivision if applicable, plus plasticity-characteristics symbols.

Summary.—In addition to reasonably satisfying requirements A to D, the features of the QPC system have been demonstrated to be sound from considerable experience with many types of work—airports, highways, dams, levees, and foundations. The use of soil names in Part 1 of this classification follows the oldest system—the descriptive classification. The grain-size numbers (Part 2A) have been used since 1937 with soils from New England to the Gulf Coast, and have been recommended by Joel D. Justin, M. ASCE (49), as well as by others who have given this system a serious trial. As a part of the Casagrande system, the plasticity-characteristics symbols (Part 2B) have been used intensively in airport construction since 1942, and are based on data collected by the author over a much longer period. Taken together, these three phases

of the QPC system reasonably cover the results of classification tests for gradation, Atterberg limits, and organic content. For judging the major soil properties, these three classification tests are considered the best currently available when used in conjunction with information on the grain shape and mineral composition. Although additional symbols could be added to cover the last two factors, the writer feels that this can be handled adequately by proper choice of the soil name.

The author is to be congratulated for the excellence of his critical review of the various classification systems, and for devising a classification which is a decided improvement over prior systems. If, from his efforts and those of others, the profession eventually succeeds in evolving a truly adequate system of soil classification, that would, indeed, be a forward step.

GEORGE F. SOWERS,[10] JUN. ASCE.—The purpose of a classification is to make it possible for materials which have similar characteristics to be considered and treated as a group. Those characteristics of immediate interest determine the grouping and the boundaries of the groups. The Airfield Classification (AC) System classifies soils by solid ingredients. Soils whose grains are coarser than 0.1 mm are classified by grain size and grain-size distribution; soils finer than 0.1 mm are classified by the liquid limit and plastic index. These properties serve as guides to the capabilities of the soil as an airfield subgrade, but they may be misleading in other applications, as implied by the author in the "Introduction."

Two properties closely linked with the performance of the soil as a foundation material are not considered in the AC system. These are the water content and the consistency. A soft clay of low "plasticity" may be a worse foundation material than a hard, dried-out clay of high "plasticity." A highly plastic clay is not highly compressible unless it has a high water content. Although the author stresses this fact when he states that compressibility must be compared at the same preconsolidation loads, further qualification should be made. Compressibility is dependent on water content; to ignore this in classification may lead to trouble. The meaning of the terms plasticity and consistency are often confused, and the inclusion of the one factor and the exclusion of the other in the AC system might make the situation worse. To a soils engineer, plasticity refers to the range in water content through which the soil is in a plastic state. However, many engineers and especially contractors and drillers use "plastic clay" to mean just what the name implies—clay in a soft or plastic state. To the man who has to dig the soil, or to build on it, the important point is the consistency of the soil, or its water content with regard to its plastic range. Ignoring these points may be dangerous, especially in the light of the confused terminology.

The greatest difficulty with all classifications is the fact that the soils are grouped according to laboratory tests. The nature of the soil deposit is usually given only secondary consideration, although it is often extremely important. For example, a varved clay consisting of alternate layers of silt and clay probably would be tested in the laboratory as a mixture of both. The classification

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tests might reveal a sandy or silty clay (perhaps CL) whereas the actual deposit may exhibit entirely different characteristics. Glacial deposits with erratic inclusions of clay, silt, and gravel defy simple classification; their variations in structure are far more important that the characteristics of each individual material.

Loess soils, which cover large areas in the Mississippi Valley, are as hard as concrete in their dry state, but are soft when saturated. These cannot adequately fit into any system of classification that depends on the index tests of soil solids in a disturbed condition. The same holds true for cemented soils, such as the red sands of the south and southwest. The density of natural sand deposits may be more important than the size or size distribution of the grains, and the presence of water only complicates the possibilities.

The discussion of the uses and of the limitations of soil classification systems by the author in the "Introduction" is a sensible approach to this problem. In order to reduce the soil groups to a finite number, many important characteristics must be ignored. Special classifications, such as the AC system applied to airfields, serve an important purpose, but their limitations should be clearly understood. Too often a classification such as this, coupled with such tabulations as Table 4, give some technicians a false sense of security.

The descriptive system in Section 15 has none of these defects because the descriptive terms can be modified to suit conditions at hand. The greatest difficulty with such a classification is a lack of standard terminology. This obstacle need not be condemnatory, for it has been encountered in other fields, such as meteorology and mineralogy. In Section 15, various degrees of each characteristic in the list of descriptive adjectives (as applied to soils, covering such points as consistency, plasticity, and gradation) could be indicated by numbers, similar to Moh's scale of mineral hardness, or to Beaufort's scale of winds. The degrees should be few so as to be distinguishable through the field tests and through the hand identification methods described in the paper, and they should be standardized by appropriate laboratory tests.

The author's plasticity chart (Fig. 4) would be well suited to standardizing the plasticity and the potential compressibility of the fine-grained soils. Consistency could be defined by the unconfined compressive strength as suggested by K. Terzaghi, M. ASCE. On the other hand, engineers are at such great odds as to the meaning of "silt" and "clay" that it probably will be better to use these terms only in a qualitative sense.

In the application of the descriptive method, both the trained soils specialist and the design engineer will have a better picture of the important characteristics of the soil, unhampered by insertion into limited pigeonholes. As Professor Casagrande points out in the "Introduction," "Those who really understand soils can, and often do, apply soil mechanics without any formally accepted classification."

RENÉ S. PULIDO Y MORALES,[11] ESQ.—The subject of soil classification, as stated by the author, is the most confused chapter in the study of soil mechanics. Most people believe erroneously that any soils classification can be

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used indiscriminately. In order to be successful, the classification must be in accord with the physical properties so that, by using the identification tests, the two most important requisites of compaction and plasticity can be found. It is necessary to know the specific gravity, apparent specific gravity, water content, mechanical analysis, Atterberg limits, and even the elementary chemical tests, so as to establish a relation between the compaction of the soil and the load applied to it.

A knowledge of these properties is important, but one must also study the performance of each soil within a given profile for similar work in other places (highways, foundations, dams, and others) so that, once the relations between the tests and their performance are established, the soils can be classified as types.

The research required to correlate the tests with the performance of the finished works can be obtained only after long periods of time. After the completion of a project the tests can be correlated with experience only by extensive research. The first and most important characteristic to be determined is cohesion, which is a function of the resistance of a soil to consolidation. Therefore, it is necessary, and most important, to include the consolidation test in the research program. The granulometric test is also important, and it must be taken into account as well as a measure of the structure as an index type.

The disintegration of igneous rocks will be more or less in an advanced state if the percentage of fine particles is more or less high. Hence, the result of the disintegration state may be determined by the granulometric curve, which is concave upward if the state is advanced, and downward if it proceeds gently. Intermediate grades are represented by a straight line.

The Public Roads (PR) classification has been used to classify for soils for highways. One of the uncertainties of this system is in group A-4 (Table 3), in which, frequently, one finds silty clays with high capillarity; but this is not the general case since silty clays and silty loams, with little cohesion and low capillarity, are also found. Another uncertainty occurs in the elimination of the material retained in the 40-mesh sieve for the determination of the physical properties and also in the exclusion of the physical determinations of the state of compaction of the soil.

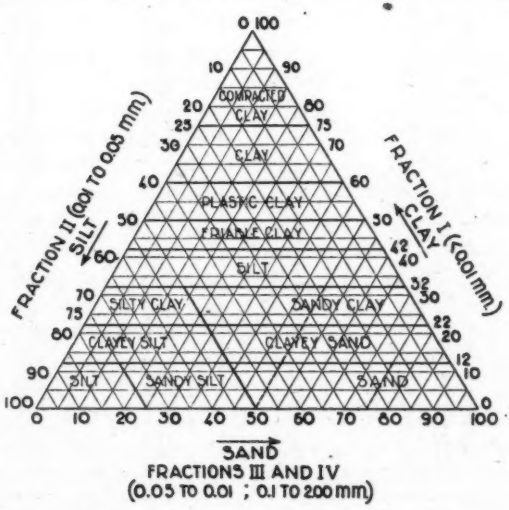

FIG. 9.—CLASSIFICATION BY SWISS ENGINEERS

The author reviews many systems of soils classification with great success. The trilinear graphs in Fig. 3 find an interesting comparison in the textural classification of Swiss engineers (50) as illustrated by Fig. 9.

The latest modification of the PR classification, presented for the Highway Research Board to its Committee on Classification of Materials for Subgrades and Granular Type Road (51), is probably the best tool yet devised for the highway engineers. This system eliminates overlapping, subdivides several groups, and introduces the "group index" which is very useful for evaluating the soil within a given group.

In this new classification the silty clay (group A-4 in the previous PR system) has been made A-6, limiting group A-4 to silty materials. The indeterminancy of the previous system has been eliminated by inserting other subgroups.

TABLE 7.—COMPARISON OF PR AND MODIFIED PR CLASSIFICATIONS

Laboratory No.	Percentage passing mesh No. 200	Liquid limit	Plasticity index	PR group	Modified PR
S-108-1662	4.9	24.7	1.9	A-1	A-1-a (0.2)
M-56-1676	37.6	21.8	0.0	A-1-2	A-4 (0.9)
C-102-560	54.4	25.0	8.9	A-2	A-4 (4.3)
H-22-668	34.5	33.1	3.4	A-2	A-2-4 (0.0)
H-22-908	36.6	46.6	8.7	A-2	A-6 (0.0)
H-22-919	36.7	0.0	0.0	A-2	A-4 (0.0)
H-41-1236	29.2	28.0	7.4	A-2	A-2-4 (0.4)
V-85-1319	2.6	30.1	4.6	A-2	A-1-a (0.0
P-14-1381	40.0	32.1	8.5	A-2	A-4 (1.0)
V-70-1401	38.0	20.4	2.6	A-2	A-4 (0.8)
H-25-1448	27.4	34.4	1.3	A-2	A-2-6 (0.6)
P-14-1465	41.8	32.7	11.8	A-2	A-6 (2.0)
M-56-1654	46.8	23.6	3.2	A-2	A-4 (0.3)
H-41-1223	59.4	33.5	7.9	A-2-4	A-4 (5.3)
H-25-1451	26.9	31.6	8.4	A-2-4	A-2-4 (0.3)
M-43-927	33.3	0.0	0.0	A-3	A-3 (0.0)
P-4-806	36.4	26.3	2.7	A-4	A-4 (0.0)
S-105-489	54.6	36.7	5.3	A-4	A-4 (4.5)
S-105-491	75.8	32.5	1.2	A-4	A-4 (8.6)
S-105-492	37.8	25.7	4.6	A-4	A-4 (0.95)
S-105-494	41.4	31.5	7.1	A-4	A-4 (1.4)
S-105-504	81.1	40.1	12.0	A-4	A-6 (9.0)
S-105-506	68.7	39.4	14.8	A-4	A-6 (8.3)
V-72-550	85.3	22.1	17.7	A-4	A-6 (11.4)
V-80-598	15.7	34.6	11.1	A-4	A-2-6 (0.2)
M-59-949	35.8	44.3	9.5	A-5	A-5 (0.0)
S-108-1051	60.1	48.4	7.56	A-5	A-5 (6.4)
H-22-634	94.8	92.5	46.6	A-5	A-7-6 (20.0)
H-22-635	84.9	89.0	43.3	A-5	A-2-5 (0.0)
H-22-636	23.8	49.6	7.8	A-5	A-2-5 (0.0)
H-22-748	70.7	51.1	14.9	A-5	A-7-5 (11.0)
H-22-749	69.4	60.0	23.8	A-5	A-7-5 (17.2)
H-22-751	73.2	74.7	30.8	A-5	A-7-6 (19.8)
H-22-778	63.6	48.2	7.3	A-6	A-5 (7.1)
C-102-844	82.2	114.0	60.3	A-6	A-7-5 (20.0)
C-102-845	8.6	64.8	33.3	A-6	A-2-7 (0.5)
C-109-891	58.2	35.0	10.5	A-6	A-4 (5)
H-41-1222	48.4	39.7	12.9	A-6	A-6 (3.66)
H-22-633	84.0	54.7	21.8	A-6	A-7-5 (15.2)
V-80-593	42.2	84.5	41.9	A-6	A-7-5 (7.1)
H-22-632	82.1	62.0	27.9	A-7	A-7-6 (18.8)
H-22-755	40.8	44.6	8.9	A-7	A-5 (1.2)
H-22-757	39.2	76.8	44.1	A-7	A-7-5 (4.8)
V-80-587	37.6	54.5	22.0	A-7	A-7-5 (15.7)

It is difficult to discuss soil classification for highway engineering purposes without mentioning the California Bearing Ratio (Porter method). This system provides the best way to determine the soil properties for a definite project and, with the help of the modified PR classification, is the more logical method. Table 7 is an example of some soils tests, made by the writer at the Soil Laboratory of the Public Works Department in Havana, Cuba,

which have been classified under the PR and the modified PR classification (51) for comparison and also to show the "group index."

In conclusion the writer wishes to congratulate Professor Casagrande on his clear presentation of this soil classification, a problem that is sorely in need of international standardization.

RAYMOND F. DAWSON,[12] M. ASCE.—A stimulating review of the various soil classification systems which have been used is presented in this paper. A new method of classification is also described which will be a real help to engineers working with soils; but, equally important, Professor Casagrande emphasizes the limitations of any soil classification system. Regardless of these cautions many engineers will adopt or continue to use a single classification system for all types of earthwork, and sooner or later this practice will cause difficulty.

A review of the soil investigations published since 1917 reveals an amazing number of classification triangles that have been proposed. It appears that every investigator wished to insure his name for posterity by attaching it to a classification triangle. Actually, these men were attempting to determine all the soils characteristics from the classification triangle, and this was impossible. Instead of seeking new information they continued to juggle old charts, looking for a combination that would be the panacea.

Not until the Atterberg limits were included in the criteria was any real progress made in the classification of soils for engineering purposes. One of the first schemes using the Atterberg limits was the Public Road (PR) classification system, of which the author correctly states (in Section 6): "This classification, therefore, is essentially a grouping applicable to the use of soil in a road surface or as a base beneath a thin bituminous wearing surface." Regardless of this limitation, highway engineers continually use the PR classification system for all types of earthwork construction. Sometime ago, an engineer called the writer's attention to a paper describing a construction method using an A-7 soil in a northern state. He said, "They cannot use that procedure with an A-7 soil; I know, because we tried it." Thereupon he visited the project and found that they were using the method as described. The soil in the northern state had a liquid limit of 42 and a plasticity index of 25, whereas the Texas soil had a liquid limit of 86 and a plasticity index of 50. Both soils were classified as A-7 by the PR system; however, they were two entirely different materials for the construction involved.

The author's Airfield Classification (AC) System is a forward step in the classification of soils for use as subgrades of pavements in airports and similar structures. The AC system will undoubtedly be the standard of classification for subgrade soils in airfield construction.

Another subgrade classification system (51) uses the group index which utilizes both the grading and the Atterberg limits of the soil. To the writer, the group index appears to offer a valuable classification system, although he is of the opinion that modifications of the limit ranges may be necessary. It will be interesting to have the author's discussion of the group index system.

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Professor Casagrande comments (in Section 10) that "Experience has proved that the liquid limit is a good measure of relative compressibility." It is of considerable importance, and further discussion of this statement is desirable.

Engineers must be cautious, or they may be misled by local terminology in describing soil formations. In the Texas gulf coast area, old-time drilling operators call the first clay stratum beneath a sand stratum a "shale" regardless of how soft it may be. Under such conditions the engineer who is unfamiliar with these colloquialisms and who does not examine the cores may not be aware of a serious condition that exists.

D. F. GLYNN,[13] JUN. ASCE.—Since a soils classification provides a uniform standard of nomenclature for field identification and for condensing engineering experience with soils, the confusion reviewed by Professor Casagrande's paper is most disturbing. Thus, immediate considerations arising from it concern: "What order could be produced in the existing position?" and "which of the systems described are suitable for incorporation in a coordinated classification?"

Any comprehensive standard classification system would have to meet the needs of agricultural scientists and geologists in addition to civil engineers. Any single system, sufficiently comprehensive to cover all these different interests, would probably be too complex for convenient use. An alternative would be the adoption of a multistage system, the first stage being a simplified field identification and the second a series of detailed functional classifications for each profession. If these were kept as simple logical systems they would form the uniform standards and allow sections with special interests to formulate additional stages to cover their own local interests. It would appear from Professor Casagrande's description that agricultural scientists and geologists already have functional classifications that meet their requirements. Thus the problem is reduced to providing a uniform field identification for general use and to formulating a standard engineering classification.

For a uniform system of field identification, a modified descriptive soil classification would seem very attractive. As a result of wartime cooperation by engineers, geologists, and agricultural scientists in the Southwest Pacific Theater, such a system has already been widely adopted in Australia. This system has sufficiently standardized soil terminology to avoid the disadvantage referred to by Professor Casagrande. It is essentially a field identification system, however, and the ultimate test of the description of any soil is the majority verdict of the experienced field surveyors. In order to keep such opinions up to date and uniform, periodic conferences are held, to which all interested persons are invited. Thus, the formal descriptive standards for adjectives and soil textures are subordinate to field usage and are periodically amended where necessary (52).

An alternative to this simple descriptive standard would be the field identification system based on the Airfield Classification (AC) System. It is difficult for anyone who has not served an apprenticeship with this system to assess its

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value in practice. Nevertheless, it would seem more complex and less flexible than the simpler descriptive type of classification. As such, it would probably be much harder to reach an international agreement on the basis of this system than with the more widely accepted descriptive classification.

As a functional engineering classification, the Texas and Civil Aeronautics Administration (CAA) systems would appear too limited in their acceptance. The Public Roads (PR) system is widely used by highway engineers, but in addition to Professor Casagrande's criticism as to the difficulty in keeping abreast of the latest "edition," its basic structure is too closely tied to highway requirements to be a good general system.

From its first appearance in Australia in 1943 the AC system has been regarded by many soils engineers as the most suitable general system. After use by the Corps of Engineers, United States Army, it is probably the most widely adopted of all systems. For these reasons the most recent revision and the discussion on the soils of the various groups are extremely welcome. It impresses one as being simple and easily understood and as following logically from the simpler field description. The principal objection offered in Australia has been due to the lack of definition in the group boundaries. It is suggested, however, that this may be a distinct advantage in that it prevents indiscriminate use by people who have no knowledge of the significance of soil test results.

If the AC system could be accepted as the basic classification to be used for samples, selected from the results of simple descriptive field mapping, it is suggested that the proposed further subdivision of classes should be limited to special applications. The general system, summarized in Fig. 4, would appear sufficient for general use and more detailed subdivision might lead to unnecessary confusion.

Another feature referred to by Professor Casagrande, the grain-size scale, might be also amended to achieve greater uniformity. In Australia an increasing number of laboratories have reverted to the international standard. At a conference of regional officers of the Division of Soils, Council for Scientific and Industrial Research, at Adelaide, Australia, in September, 1946, the sand-silt size boundary of 0.02 mm was discussed (52), in the light of the proposal by R. Glossop and A. W. Skempton (5). Although no change was recommended, it is believed that change to 0.05 mm or 0.06 mm would be readily accepted if sufficiently valid reasons could be offered. On other hand, it is understood that the United States Department of Agriculture is by no means irrevocably committed to 0.05 mm. The time would thus seem ripe for discussion to achieve uniformity on at least this one point.

In view of the proposed international soil conference in Holland, Professor Casagrande's able summary of the present soils classifications is most timely. It is to be hoped that this favorable opportunity for using Professor Casagrande's wide experience in soils mechanics, soils classification, and international cooperation with engineers will not be allowed to pass.

In conclusion, it is desired to thank Professor Casagrande for the comprehensive review of the present state of soils classification and particularly for his discussion on the AC system.

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Correction for *Transactions*.—In October, 1947, *Proceedings*, page 1311, Fig. 7, change "rock flower" to "rock flour."

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DISCUSSIONS

THE GENERAL PRINCIPLES OF HIGHWAY TRANSITION CURVE DESIGN

Discussion

BY CARL F. MEYER

CARL F. MEYER,[12] M. ASCE.—Because they lead to a suggestion for a new method of transition curve design, Mr. Leeming's experiments appear to throw some light on a perplexing problem. The author's statement (under the heading, "9. Conclusion,") that "The whole argument of this paper runs counter to established practice" should, in itself, be enough to stimulate discussion. However, the phrase "established practice" may be misleading; considering all the interrelated factors, there is hardly any established practice, at least in the United States. This fact may be noted by examining the recommendations of such bodies as the United States Public Roads Administration and the various state highway departments in the United States, wherein marked variations will be found in the length of transition and in the superelevation rate for identical conditions of radius and design speed.

It is true, however, that most attempts at rationalizing this problem have been based on the use of some constant rate of turning, C; but no conclusive evidence exists as to whether C should be 1, 2, or some other value. Therein lies an important reason for the great variation in transition lengths that are found.

The principal objection which the author has to the previous theory (see "Synopsis"), is its "disregard of the superelevation." Mention is made (under the heading, "3. The Rate of Turning the Steering Wheel"), for example, of F. G. Royal-Dawson's method which

> "* * * contains the fundamental fallacy of using C and the centrifugal ratio, ignoring the superelevation. The theory gives the same length for a transition curve, no matter what the value of the superelevation."

Although there may be a fallacy, it should be emphasized that in the previous theory superelevation is not completely ignored or disregarded. Present prac-

NOTE.—This paper by John Joseph Leeming was published in October, 1947, *Proceedings*.

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tice bases superelevation on R, according to some variable allowance for side friction and up to some practicable maximum. The length of transition is then based on R, design speed, and some value of C. Joseph Barnett,[13] M. ASCE, for example, suggests $C = 2$, giving $L = 1.6 \frac{v^3}{R}$. Thus, L is dependent on the superelevation, although possibly not in accordance with correct dynamic theory.

There is little doubt that the crux of the problem lies in conclusive answers to the two questions posed by the author in Section 3. He is to be commended for his ingenious efforts to answer them. After studying Mr. Leeming's paper, describing his experiments and the resultant conclusions, the writer suggests that answers to the following might aid in forming a better understanding of the experimental procedure and in judging the validity of the conclusions:

(1) Can the author supply a sketch of the accelerometer setup and a copy of a typical (Q–j)-graph?

(2) What were the ranges of values of Q and j corresponding to the four groups of comfort experienced by the observer? It would be interesting to compare values of j with the side friction factors found by R. A. Moyer,[14] Assoc. M. ASCE, and Mr. Barnett.[15]

(3) Were the comfort sensations those of the same driver and observer, or were different pairs of individuals used?

(4) Did the test curves traversed include those with transitions as well as those without; and, if so, did this difference affect the results?

(5) If, as stated (under the heading, "5. Experimental Procedure"), "* * * no attempt was made to separate the sensations due to the force or to its rate of change," how was it possible to arrive at the conclusion (under the heading, "6. Conclusions from the Experiments") that "(4) As far as comfort is concerned, j is the important effect; and Q is unimportant?"

The author was frank in mentioning the limited scope of his experiments imposed by wartime restrictions on speeds. The availability of only one American car was unfortunate, notwithstanding the author's finding that the type and the weight of vehicle seemed to have little effect on the results.

The finding (under the heading, "5. Experimental Procedure"), stating, "Another fairly marked tendency was for the driver to use a definite proportion of the curve as transition, * * *" seems quite significant provided that many different drivers were involved. This tendency, taken in connection with the mathematical analysis represented by Fig. 3, is substantial argument against large values of p, and especially devastating to the all-transitional curve favored by some engineers. Assuming that these and later extensive experiments justify the author's conclusions, the practical application still remains to be developed. In doing this the author seems to favor the "proportion rule"

[13] "Transition Curves for Highways," by Joseph Barnett, U. S. Bureau of Public Roads, U. S. Govt. Printing Office, Washington, D. C., 1938.

[14] "Skidding Characteristics of Automobile Tires on Roadway Surfaces and Their Relation to Highway Safety," by R. A. Moyer, *Bulletin No. 120*, Iowa Eng. Experiment Station, Ames, Iowa, 1934.

[15] "Safe Side Friction Factors and Superelevation Design," by Joseph Barnett, *Proceedings*, Highway Research Board, National Research Council, Washington, D. C., Vol. 16, 1936.

(Section 8), but apparently only "for the moment." Is his reticence due to the feeling that something better will be found, or is he doubtful as to the proper value of p to use?

Although the proportion rule is easy to apply, a moderate variation in the proportion of transition results in a rather large change in transition length. For instance, in the example worked by the author in Section 8, using $p = \frac{2}{3}$, the resulting values of R and L were about 2,570 ft and 494 ft, respectively. A similar computation with all data unchanged except $p = \frac{1}{2}$ will give approximate values for R and L of 2,670 ft and 350 ft. The degree of curve remains about the same, but the transition length is reduced about 30%. As the author noted (under the heading, "8. Alternative Method of Transition Curve Design"), "The exact length of the transition is not, however, a matter of extreme importance * *.*." Apparently flexibility in the matter of transition length is obtained in the proportion rule method by varying the value of p.

In the practical application of the proportion rule, the values of R and L found by using a particular value of p would be slightly altered for convenience in staking (at least by most engineers in the United States). In the example given in Section 8, for instance, the computed values were $R = 2{,}570.93$ ft (or $D = 2°13' +$) and $L = 493.58$ ft. Probably 500-ft spirals joining a 2°10′ or 2°15′ curve would be used. Of course, E would not then be exactly 55 ft, but a rigid requirement permitting no deviation from a fixed E is rarely met.

The tables of spiral functions (Table 1), for which the equations use the radius as parameter, are very useful for the special cases involving a maximum value of E or T. Since these situations happen frequently, it is to be hoped that engineers will not reject the author's proportion rule solely because of the absence of such special tables in route surveying textbooks, or among the tables issued by the American Railway Engineering Association (AREA). This omission is not a stumbling block, for the usual AREA ten-chord tables and formulas may be used with only slight added effort. A quick slide rule solution of the example in Section 8 demonstrates this fact.

First, solve Eq. 8*e* for $\phi = 5.7°$ and round off to 5°30′ as in the given example. (Incidentally, it should be realized that Eqs. 8*c* and 8*e* apply also to the AREA spiral, and may be derived by using the basic relation $\phi = \frac{L\,D}{200}$.)

Then, in the AREA formula—

$$E = (R + o)\ \text{exsec}\ \frac{\Delta}{2} + o \qquad (12a)$$

—there are two unknowns (R and o); but since the length o is always small by comparison with R it may be neglected temporarily and R solved for (approximately. $\frac{E}{\text{exsec}\ \frac{\Delta}{2}} = \frac{55}{0.01983} = 2{,}770$ ft). Thus (from AREA tables) D = approximately 2°04′ and $o = 4.25$ ft. Next, recompute a closer value of R from

$$R + o = \frac{E - o}{\text{exsec}\ \frac{\Delta}{2}} \qquad (12b)$$

giving $R = 2{,}555$ ft. Then $D = 2°14'$ (or $2°15'$). Finally, L (from Eq. 8c) = 492 ft. In practice this would probably be rounded to 500 ft, using ten 50-ft chords.

Despite the author's objections to the current theory, certain of the methods that contain the several objections he has raised give results surprisingly close to the method he now proposes, as may be illustrated by simple computations using the Barnett tables.[13] In the Barnett method full superelevation is computed for 75% of the design speed up to a maximum rate of 0.10 ft per ft. The length of transition approximates that obtained from—

$$L = 1.6\frac{v^3}{R} \qquad (13)$$

—which is derived by using $C = 2$. The Barnett tables are built for values of degree of curve varying by 30′ or 1° increments, and transition lengths varying by 50-ft or 100-ft increments. In Table 3 several combinations of D and L are given which fit closely the conditions of the author's example, namely, design speed = 80 miles per hr, $\Delta = 22°38'$, and $E = 55$ ft (approximately). Values of D, L, T, and E were obtained quickly by simple interpolation from the Barnett tables; the values of ϕ, p, e (superelevation rate), and j have been computed for comparison with the author's values in Section 8. It will be observed that the combinations using 350-ft and 500-ft transitions compare closely with the values obtained by the "proportion rule" using $p = \frac{1}{2}$ and $p = \frac{2}{3}$, respectively. The largest difference seems to be in the matter of superelevation rate, the values of which could be scaled downward toward the author's recommendations (with consequent increase in j) without changing the values of the other quantities in Table 3.

TABLE 3.—SUITABLE TRANSITIONS FOR EXAMPLE OF SECTION 8

D	L	T	E	ϕ	p	e	j
2°00′	300	723.6	48.2	3.0	0.42	0.084	0.065
2°00′	350	748.7	58.6	3.5	0.47	0.084	0.065
2°30′	400	659.2	48.4	5.0	0.61	0.10	0.086
2°30′	500	709.5	51.1	6.25	0.71	0.10	0.086
2°30′	600	759.8	52.1	7.5	0.80	0.10	0.086

It was not the writer's intention to show the preceding comparison for the purpose of discounting the author's alternative method of transition design. On the contrary, it is felt that the author has made a genuine contribution to this interesting problem. As for the proposed method (under the heading, "9. Conclusion"), taking "* * * the whole problem back to rule-of-thumb days * * *," there appears to be no more, if as much, rule of thumb than exists in the present theory. Even though the author feels that his method may meet resistance because it runs counter to present theory, the writer is encouraged to feel that engineers are willing to reject anything found unsound (as stated in the "Conclusion") in favor of "* * * sound reasoning supported by experimental analysis * * *." The modern engineer does not prefer old-established error to novel truth.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

ESTIMATING DATA FOR RESERVOIR GATES

Discussion

BY THEODORE B. RIGHTS

THEODORE B. RIGHTS,[6] ASSOC. M. ASCE.—In this discussion the writer will confine his remarks to radial crest gates. Unless the stress used is noted on the design of the structure a person who is seeking to compare his structure with the one previously designed, is "in the dark." In comparing a newly designed radial gate with an older gate that had a head 5 ft less, the writer was amazed that his designed sections were smaller in size. On investigation he found that, although the skin or face plate of this older gate was designed for 16,000 lb per sq in., the supporting members were only stressed to a maximum of 10,000 lb per sq in., whereas the writer was using 18,000 lb per sq in. throughout. It would be worthwhile if Mr. Boissonnault were to qualify his charts by taking the design stresses into account. For example, the following equation for the weight of the radial or Tainter type of gates[7] makes allowance for unit stress:

$$W = \frac{935\,B\,H_d}{\sigma}\,(A^2_s + 700) \qquad (6a)$$

and

$$w = (A_s + 2\,B)\,125 + 225\,L \qquad (6b)$$

in which, conforming to the notation of the paper: W is the weight of the gate (moving parts), in pounds; w is the weight of the embedded metal, in pounds; L is the length of gate lift, in feet; B is the height of the gate, in feet; H_d is the design head to the bottom of the gate measured in feet; σ is the design stress of the material, in pounds per square inch; and A_s is the clear span width of the gate, in feet.

(Incidentally, it may not be amiss to re-emphasize the widespread misspelling of the word "Tainter." Although the author spells it correctly, it appears as "Taintor" even in some of the most modern technical textbooks. The records of the United States Patent Office for July 6, 1886, show, under "canal lock," that patent No. 344,879 was registered by Jeremiah Burnham

NOTE.—This paper by Frank L. Boissonnault appeared in September, 1947, *Proceedings*.

[6] Cons. Engr., Roselle, N. J.

[7] "Engineering For Dams," by W. P. Creager, J. D. Justin, and Julian Hinds, John Wiley & Sons, Inc., New York, N. Y., Vol. III, 1945, p. 906.

Tainter of Menomonie, Wis. The device therein described is the well-known Tainter gate.)

Many of the gates used by the power companies are adaptations of old gates, in which the stresses at the time of their design were 16,000 lb per sq in. One company, specializing in hydroelectric design, does not have a gate in their files that has been designed for 18,000 lb per sq in. When this company receives an order from a client company to design a gate, they search their files, select one of these gates (16,000 lb per sq in), secure "photolithos," and block out titles and elevations. In fact, one of the greatest difficulties in doing this is to detect the old sections that have not been rolled in the past few years. This company should not be overly criticized for using these uneconomical designs as the steel detailed drawings have been made. Photolithos of these details serve the shop just as well as new details, and the cost of making the shop drawings is thereby reduced about 80%, which should give the client company a cheaper job than if a new design with new detail drawings were used. Of course, in order to do this the contract for the gate must be given to the original fabricator.

The writer has used the charts by A. E. Niederhoff[5] and the results approximate closely the weights obtained from the writer's designs. These weights are for the gate only and not for the anchorage which comprises the larger part of the embedded steel. For an estimate for the larger gates, the writer uses 60% of the gate weight for the weight of the embedded steel. The weight of the anchorage is not directly a function of the gate weight. The location of the pin, both horizontally and vertically, will change the hinge reactions. For identical gate widths and heights, but with different pin locations, different anchorages may be required. For a small hinge reaction, the pier width is usually narrow. Most installations of these small gates have a continuous pin that runs through the pier, so that one arm of one gate and one arm of the adjacent gate rotate on the same pin. This pin is held back by pin plates with anchor rods bolted to them, and the entire assembly is grouted into the pier with only the pin ends cantilevering out. The main anchor rod can be placed in the line of action of the hinge reaction. For gates more than 33 ft wide with a 30-ft head, the piers must be much wider for stability. It would be impracticable to run the pin through the piers, so an anchorage casting placed on cantilever beams is used most frequently. These cantilever beams, both horizontal and vertical, are held down by anchor rods. Since these anchor rods are horizontal and vertical, they must take the components of the hinge reactions, and they are larger, therefore, than the anchor rods placed in the line of the hinge reaction. This type of construction increases the weight of the anchorage considerably. The writer believes that this in part explains the discrepancy between the actual weights and the weights obtained from Eq. 6*a*.[7]

The weight of the embedded steel as well as the hoist weight may vary because of the nature of the installation. If a music-note seal (such as that used on some of the Tennessee Valley Authority radial gate installations) is used instead of a wood seal, the friction of the seals is increased and the hoist may have to be increased for this item. In some climates, as stated by the

[5] "Selection of Tainter Gates," by A. E. Niederhoff, *Western Construction News*, December, 1943, p. 555.

author, the weight of ice and silt on the gate is considerable. The writer has found that it is better actually to compute the hoist requirements, than to use any published data.

The writer further believes that a distinction should be made in this paper between fixed and movable hoists. Some dam installations have been made in the United States, in which a movable or traveling hoist car is used for three or four gates. Of course, a suitable structure must be provided for this movable hoist but, as it is made of structural steel, its cost can more easily be computed.

The writer believes one of the reasons that the welded gates weigh less than the riveted ones is that they, being more modern, have been designed for the higher stress of 18,000 lb per sq in.

TABLE 6.—WEIGHTS OF RADIAL CREST GATES USED IN POWER DAMS AND WATER SUPPLY DAMS

(Designed for a Stress of Less Than 16,000 Lb per Sq In.)

Gate No.	Date built	Feet				Kips							
						Actual Weight							
		Radius of skin plate	Height of pin above crest	*A*	*B*	Gate only	Embedded steel	Total	Boissonnault	Niederhoff	Eq. 6a; Gate only	Eq. 6b; Embedded steel	Total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	1930	32.0	16.0	39.0	30.0	110.0	65.0	175.0	160.0	120.0	117.0	19.1	136.1
2	1948	35.0	18.0	39.0	35.0[a]	145.0	88.0	233.0	200.0	150.0	159.0	21.5	180.5
3	1930	30.0	24.0	34.0	26.0	83.0	....	....	110.0	75.0	73.0	16.7	89.7
4	1929	26.0	14.0	34.0	26.0	62.0	....	....	110.0	75.0	73.0	16.7	89.7
5	1926	26.0	14.0	34.0	24.0	62.0	....	....	92.0	71.0	62.5	15.7	78.2
6	1944	20.0	14.75	33.0	19.0[b]	36.0	11.5	47.5	53.0	42.0	30.3	12.2	42.5
7	1937	24.0	20.0	30.0	16.0[c]	24.0	7.5	31.5	40.0	31.0	21.0	11.3	32.3
8	1925	18.0	7.0	30.0	14.0	28.0	12.5	40.5	36.0	28.0	18.4	10.5	28.9
9	1927	13.0	8.0	24.0	10.0[d]	11.0	1.5	12.5	18.0	14.5	9.0	8.3	17.3
10	1916	20.0	13.0	20.0	13.5[e]	13.0	2.5	15.5	19.0	15.0	11.7	9.0	20.7
11	1913	17.0	9.0	15.0	14.0	9.0	....	....	13.0	12.0	10.5	8.6	19.1

[a] Actual head 40.0 ft. [b] Actual head 17.0 ft. [c] Actual head 15.0 ft. [d] Design head 11.0 ft. [e] Design head 14.0 ft.

The values obtained from the charts in this paper are useful; but it must be remembered that they should be used for estimating purposes, rather than as a rigid control on the design weight. Discussing this point, James S. Bowman, M. ASCE, states:[8]

> "The general layout of a Tainter Gate is a matter of trial and judgment since there is no fixed relation between the length of the arms and the height of the gate. The pins should be above ordinary high water, and the bottom of the gate when raised should clear floating material carried by the extreme flood. The shorter the radius of the face or the greater the inclination of the chord from the vertical, the heavier the gate will be."

Elsewhere it is stated:[9]

> "The radius of curvature affecting the direction of this upward pressure (upward vertical component of the water pressure), also has an important

[8] "Handbook of Applied Hydraulics," McGraw-Hill Book Co., Inc., New York and London, 1942, p. 367.

[9] "Manual of Engineering Construction," Canalization, Vol. II, The Engineer School, U. S. E. D., Fort Belvoir, Va., 1940, p. 289.

bearing on the relative dead load reaction at the chain and trunnion, on the length and height of the pier, and the weight of the struts."

From the foregoing it can be seen that two gates of identical widths and heads can be of different weights because of the different radii to the face plate and because of the different heights of pins above the crest. A good example of this can be seen in gates 3 and 4, Table 6. With the exception of gate No. 10, the curved skin plate is supported by horizontal channels variably spaced which in turn are supported by vertical ribs that frame into the trunnion arms. Gate No. 10 has the curved skin plate supported on equally spaced vertical ribs which are in turn supported by horizontal beams that frame into the trunnion arms.

For the writer's work, it would be best to have the gate weight separated from the total weight of the gate and the embedded steel. The charts do not give the height of hoist or lift as a variable. In canal work, most gate installations will have approximately the same lift, but in power or water dams, where large flows of water must be passed so as to have a constant head on the turbines or pumps, the lift of gate must be peculiar to that installation.

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DISCUSSIONS

TESTS OF TIMBER STRUCTURES FROM GOLDEN GATE INTERNATIONAL EXPOSITION

Discussion

BY SIDNEY NOVICK, FRANK J. HANRAHAN, AND WALTER J. RYAN

SIDNEY NOVICK,[20] JUN. ASCE.—It is of interest to compare the results of the plywood shear buckling tests (under the heading, "IV Plywood Tests; Group 3") with the values at which failure in buckling might have been predicted on the basis of previously published information concerning shear buckling of rectangular plywood plates. The method and results of this comparison are described below.

All calculations require basic assumptions regarding the specific plywood constructions. From the descriptions in the text, it was inferred that plywood type 1 was of the sanded face ply variety, and that types 2 and 3 were unsanded; the thicknesses of the individual plies could then be found from data issued by the Douglas Fir Plywood Association.[21] Assuming the modulus of elasticity along the grain for a single ply to be 1,600,000 lb per sq in. and that across the grain to be 0.045 times the value along the grain,[22] all necessary plywood stiffness properties may be found.

A convenient method for calculating the theoretical buckling stress of a simply supported rectangular plywood plate under shear has been presented elsewhere.[23] Later empirical investigation[24] (with plates acting as webs of box beams) indicated that the theoretical stresses require correction within a given range of plate dimensions. From the empirical data the writer has derived

NOTE.—This paper by a Committee of the San Francisco (Calif.) Section, ASCE, on Timber Test Program was published in May, 1947, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1947, by James H. Carr, Jr., Howard J. Hansen, and E. G. Stern; and November, 1947, by Melvin W. Jackson, and Charles Mackintosh.

[20] Structural Dept., John B. Pierce Foundation, Housing Research Div., Raritan, N. J.

[21] "Technical Data on Plywood," Douglas Fir Plywood Assn., Tacoma, Wash., p. 9, Section 2.

[22] "Design of Plywood I-Beams," by Howard J. Hansen, *Transactions*, ASCE, Vol. 112, 1947, p. 955.

[23] "Buckling of Flat Plywood Plates in Compression, Shear or Combined Compression and Shear," *Bulletin No. 1316* and Supplements, Forest Products Laboratory, Forest Service, U.S.D.A., Madison, Wis., 1942–1945.

[24] "Design of Plywood Webs in Box Beams," *Bulletin No. 1318* and Supplements, Forest Products Laboratory, Forest Service, U.S.D.A., Madison, Wis., 1943–1944, Fig. 16.

a curve[25] from which the required correction factors can be obtained. Specific values necessary to derive factors needed for the ultimate buckling stress calculation were also obtained from other published data.[26] Values given in Table 24 were used as the maximum plywood shear stresses in these calculations.

The comparison of results of the foregoing calculations with the actual shear stresses at failure, the latter derived from Table 25, is given in Table 29.

TABLE 29.—COMPARISON OF PREDICTED SHEAR BUCKLING STRESS AND ACTUAL SHEAR FAILURE STRESS IN PLYWOOD

Specimen No.	Stress (Lb per Sq In.)		Variation of actual from predicted stress (%)
	Predicted shear buckling	Actual shear failure	
(1)	(2)	(3)	(4)
(*a*) Failure Due to Buckling			
1	135	370	+174
2	170	330	+ 94
4	265	395	+ 49
10	220	475	+116
11	340	375	+ 11
Average	...	...	+ 89
(*b*) Failure Due to Nails			
3	240	320	+ 33
5	405	440	+ 9
6	580	395	− 32
7	370	395	+ 7
8	580	340	− 41
9	850	440	− 48
12	395	410	+ 4
13	295	335	+ 11
14	445	340	− 24
15	615	300	− 51
16	370	500	+ 35
17	605	350	− 42
18	865	420	− 51
Average	...	...	− 15

This tabulation shows that the scatter in the percentage variation values is large for the specimens that failed by buckling. That this behavior is typical is evident from a consideration of the Forest Products Laboratory (FPL) results:[23] For the range of plate dimensions in Table 29(*a*), the FPL data show a variation between 50% and 150% greater than the values on the empirical buckling curve (the latter, of course, taken along the lower limits of the range of values). It appears that the average fixity of plywood plates when glued or nailed sufficiently to insure shear buckling is well above the simply supported condition, but that the variations in the extent of fixity lead to large variations, not only between the actual stresses and those predicted, but also in the extent of the disagreement, so that the actual values are not in constant relation to the theoretical.

From the results for the specimens in Table 29(*a*), the specimens in Table 29(*b*) might have been expected to fail in shear buckling at values proportionately above the predicted buckling stresses. It is evident that the actual failure of the nails at much lower values forestalled development of the requisite buckling stresses. Thus, the results for the specimens in Table 29(*b*) are in qualitative agreement with the behavior expected on the basis of the previously published information.

Results of the shear buckling tests reported in the paper, therefore, tend to confirm previously published data and the general method for design cal-

[25] *Transactions*, ASCE, Vol. 112, 1947, p. 968, Fig. 5.

[26] "Design of Wood Aircraft Structures," *Bulletin No. 18*, Army-Navy-Civil Committee on Aircraft Design Criteria, Washington, D. C.

culation derived therefrom. The general method, in turn, is applicable within the entire practicable range of design variables, the latter including the spacing of studs and intermediate blocking, and the thickness of plywood sheathing.

It should be particularly noted that by this method the value of the intermediate blocking can be found in explicit quantitative terms, making it necessary to alter the implied conclusion in the text that the value of such blocking is open to question.

FRANK J. HANRAHAN,[27] M. ASCE.—Those closely associated with this enormous, complicated test program are to be congratulated, both for their unceasing efforts in the face of great obstacles and for the important contribution made to the knowledge of timber engineering. These investigations, together with extensive research by the United States Forest Products Laboratory (Madison, Wis.), the Timber Engineering Company (Washington, D. C.), and others have resulted in substantial changes and technical improvements in engineering design recommendations. Such modifications are reflected in a design specification[9] which has been widely adopted as a design standard by government agencies and many other organizations. Among the items which were more accurately evaluated and are incorporated in this specification are the effect of end distances, spacings, edge distances, thickness of member, angle of load to grain, moisture condition of lumber at time of fabrication and in service, and loads for multiple fastenings—all in relation to the basic design capacity of connectors.

Among other things, the analysis of these 1941 tests, and the report on them, focused attention on the need for further study of eccentrically loaded joints and the cross-grain stresses resulting from such eccentric loading. In spite of studies conducted subsequent to analysis of these tests, no satisfactory criteria for designing eccentric joints and taking care of the resulting stresses had been developed prior to the issuance of the "National Design Specification for Stress-Grade Lumber and Its Fastenings" in 1944. There appeared to be little correlation between values obtained from standard tension perpendicular to grain tests and values obtained from tests of eccentric joints which presumably were loaded in cross-grain tension. Consequently, the aforementioned specification recommended avoidance of eccentric joints.

Subsequently extensive research studies developed an empirical approach which when applied to a wide range of tests showed a proper relationship between allowable design loads and test loads at failure. This recommendation, which currently is officially incorporated as an amendment in the national design specification and which assumes the mathematical form of the horizontal shear formula but with allowable shear stresses 50% greater than otherwise permitted, may be stated as follows:

> "In computing the horizontal shear in eccentric joints the depth of the member shall be assumed as reduced by the distance between the unloaded edge of the member and the nearest edge of the connector. Where bolts

[27] Chf. Engr., National Lumber Mfrs. Assn., Washington, D. C.

[9] "National Design Specification for Stress-Grade Lumber and Its Fastenings," National Lumber Mfrs. Assn., Washington, D. C., 1944.

alone are used, the depth shall be reduced by the distance between the unloaded edge and the center of the bolt. Shearing stress for joint details may be taken as 50% greater than the horizontal shear values otherwise permitted."

In addition, the stresses induced by axial loading, if any, and bending moment should be calculated and taken care of in the design in the customary manner.

The effect of these technical advances through the incorporation of the additional technical findings of recent years in current recommendations[28] is indicated by the values in Table 26.[28a]

When the structures tested were designed for the 1939 Golden Gate International Exposition, in San Francisco, Calif., the designer did not have access to design recommendations later than those of 1935, which were considerably less refined than those of 1939. Design stresses for lumber and design loads for fastenings higher than those commonly used in 1935 were used in the Exposition buildings. John J. Gould,[29] M. ASCE, chief structural engineer for the Exposition, has stated:

"Split rings gave satisfactory service despite working stresses 30 to 50 per cent more than usual. * * *

"The wholly satisfactory performance of exposition structures, designed with 50 to 100 per cent increases of the usual timber unit stresses, suggests that where competent engineers make the design and supervise construction and maintenance, building codes should be revised to provide for increased unit stresses—perhaps 30 to 50 per cent more than now permitted."

It should be noted that the ratios given in the report and in discussion approach true safety factors for those instances where failure occurred. The so-called factor of safety as commonly used is not a true factor of safety for any material in the sense that the load required to produce failure of the completed structure is the design load times the factor. This so-called factor of safety contains allowances for variability of material, inaccuracies of design and fabrication, and other factors. When the structure is tested to failure, many of these items are incorporated physically in the structure itself, instead of as allowances in the so-called factor of safety. Consequently, the true factor of safety as determined by test of the structure logically should be lower than the factor of safety as commonly stated.

It is interesting to note from Table 26[28a] that, for those joints in which failure is approached under test load, the lowest ratio of test load to design load is approximately 3 when based on current design recommendations[28] and is approximately $1\frac{3}{4}$ when based on those for 1939. Hence, these particular data indicate that present design recommendations actually result in structures approximately 70% stronger than those based on 1939 recommendations, in spite of the fact that the current basic allowable design values for lumber and fastenings are 20% higher than in the 1939 recommendations.

[28] "National Design Specification for Stress-Grade Lumber and Its Fastenings (1944) Including 1947 Amendments," National Lumber Mfrs. Assn., Washington, D. C., 1947.

[28a] *Proceedings*, ASCE, October, 1947, p. 1324.

[29] "Experience in Maintenance of Large Timber Structures," by John J. Gould, *Engineering News-Record*, August 15, 1940, pp. 72–74.

Although the writer does not intend to imply that such a percentage increase in strength can be expected generally, the data do indicate that improvements in design recommendations do offset increases in allowable design stresses in so far as total strength of structure is concerned. The data also illustrate the greater technical accuracy and reliability of the design recommendations incorporated in the aforementioned design specification[28] and the increased safety of structures conforming to this current specification.

WALTER J. RYAN,[30] M. ASCE.—The Timber Test Committee of the Society's San Francisco Section deserves the thanks of the profession for publishing this report of its project, hindered as it was by many obstacles that threatened to make its completion impossible. We are fortunate to have this record, which marks a big step in the progress of timber construction from an art to a science.

Builders of the ninteenth century carried forward the knowledge of timber construction that had been accumulated since the beginning of its development. They were artisans, and the structures they built were erected by men trained and interested in their work. Following the introduction of newer materials of construction, engineering methods became an essential part of the development. Now that timber engineering is becoming a science instead of an art, it is suffering from a lack of these artisans of former generations and from the lack of understanding of a common material. The very name "lumber," so frequently used in the United States, is an indication of the lack of respect for a familiar product. This report illustrates clearly how intricate a material wood really is, and the designer of timber structures should keep the ten notations, made in the "Foreword," always in mind. These findings[31] support the "ten commandments" of a west coast association.

Most of the grief in design and construction in timber is caused by a lack of uniformity of strength within the piece due to "grain," changes through "seasoning," and variation within and between "grades." These tests will supplement laboratory studies and leave no excuse for the designer to notch a timber or neglect an analysis of the various stresses acting on a member. Lumber grading rules are nearly as perfect a guide as human interpretation makes possible. However, some pieces may be unsuited for the specific application. The framer must have the knowledge and the authority to cull such items and to select material suitable for the particular purpose. The report illustrates instances where this was not done. The principal difficulty came from shrinkage and checking. This problem is inherent with the use of green timber and seems to be without a solution. The answer is in the use of seasoned material.

Air seasoning of large timbers is not practical. Drying in a kiln is difficult and expensive. Modern practice can secure the most satisfactory results by treatment in a retort. This can be accomplished with or without the addition of preservative. Such trusses as those used in the Treasure Island Buildings will be built of seasoned or treated material in the future.

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[30] Weyerhaeuser Timber Co., Tacoma, Wash.

[31] "Ten Essential Standards for Fabricated Timber Structures," West Coast Lumbermen's Assn., Portland, Ore., 1946.

The design of the first heavy trusses built in the United States with split rings was based on the tables published by the manufacturer, Carl Tuchscherer,[32] of Breslau, Germany, supported by a limited series of tests made at the University of British Columbia in Vancouver. Later tests proved that the values assigned, especially to the large 9-in. and 11-in. rings, were excessive, but that the bridges served adequately for railroad loading until the timber was destroyed by decay. The bolts in these trusses were kept tight.

In the San Francisco tests, the committee saw fit to test the joints in their condition in the structure. This condition reflected the result of shrinkage and little or no maintenance. There was lack of uniformity and, in the case of loose bolts, there was the loss of the frictional resistance which is an important element in the strength of a timber joint. Perhaps these results will help to emphasize the importance of continuous inspection and maintenance. A more valuable result will be attained, however, when adequate tests are made with perfect examples and engineers, builders, and owners are taught to treat the design and the structures with the care they deserve.

[32] "Der Holzbau," by Theodor Gesteschi, Springer, Berlin, 1926.

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DISCUSSIONS

CONSOLIDATION OF FINE-GRAINED SOILS BY DRAIN WELLS

Discussion

BY KENNETH S. LANE

KENNETH S. LANE,[15] ASSOC. M. ASCE.—As the drain well promises to become a very useful tool in foundation engineering, the author's development of the theory of consolidation with drain wells is particularly timely. The process of consolidation involves the expulsion of water from the soil voids, thus permitting the grains to attain closer contact under the influence of applied loads. In a homogeneous material the time required for consolidation varies as the square of the distance the water must travel. Insertion of sand-filled drain wells in a foundation shortens the water escape path and thus greatly accelerates the rate of consolidation. With water escaping radially to the wells, in addition to escaping in the normal vertical direction, the time of consolidation can be reduced to a fraction of that for vertical flow only (see Fig. 6) —governed mainly by the choice of well spacing. This benefit from drain wells is far greater in a horizontally stratified material (as the author has noted) where the horizontal or radial permeability is generally much larger than that in a vertical direction.

Benefits of Drain Wells.—There are three major benefits from using drain wells in a weak compressible foundation: (1) Acceleration in the rate of settlement; (2) acceleration in the rate of gain of shear strength; and (3) reduction in the lateral transmission of excess pressure.

1. *Acceleration in Rate of Settlement.*—The percentage of ultimate settlement at a given time is the same as the percentage of consolidation.

2. *Acceleration in Rate of Gain of Shear Strength.*—Let σ_t be the total unit stress on an element of soil; σ_o be the initial stress; $\Delta\sigma$ be the increment of stress added to σ_o; $\bar{\sigma}$ be the intergranular pressure; and σ_f be the final intergranular pressure. In the notation of the paper $z\,\gamma_w$ is the hydrostatic pressure and u is the pore water pressure in excess of hydrostatic pressure. The total stress on an element of soil consists of intergranular pressure plus water pressure. With the

NOTE.—This paper by Reginald A. Barron was published in June, 1947, *Proceedings*.

[15] Chf., Soils and Geology Branch, Garrison Dist., Corps of Engrs., Bismarck, N. Dak.

addition of a new load to a soil which has reached full consolidation under its former loads, this results in:

$$\sigma_t = \sigma_o + \Delta\sigma + z\,\gamma_w \quad \ldots\ldots (89a)$$

$$\sigma_t = \underbrace{\bar{\sigma}_o + u} + z\,\gamma_w \quad \ldots\ldots (89b)$$

$$\sigma_t = \bar{\sigma}_f + z\,\gamma_w \quad \ldots\ldots (89c)$$

Under a common hypothesis for the shear strength of clays, only the intergranular pressure, $\bar{\sigma}$, is effective in creating shear strength from internal friction. The process of consolidation involves the transfer of added stress, $\Delta\sigma$, initially carried in the pore water as excess pressure, u, to intergranular stress with resulting gain in shear strength. Eqs. 89*b* and 89*c* represent the beginning and end of consolidation, the hydrostatic portion of the pore-water pressure, $z\,\gamma_w$, remaining unchanged.

As the author has indicated, the factors governing the shear strength of clays are not well understood. The other common approach is based on empirical discoveries that shear strength varies inversely with water content. Under either approach shear strength increases as consolidation progresses by decreasing both pore pressure and water content.

3. *Reduction in Lateral Transmission of Excess Pressure, u, in the More Pervious Layers of Stratified Soil.*—Beneath the center of an embankment the added stress, $\Delta\sigma_c$, and the resulting excess pressure, u_c, reach their maximum values. If the horizontal permeability is much greater than the vertical permeability, as in the varved clay pictured in Fig. 9, a portion of the center line excess pressure, u_c, will be transmitted laterally to the zone beneath the embankment toe. For such conditions at the toe, Eqs. 89 become:

$$\sigma_t = \sigma_{ot} + \Delta\sigma_t + z\,\gamma_w \quad \ldots\ldots (90a)$$

$$\sigma_t = \underbrace{\bar{\sigma}_{ot} + f(u_c)} + u_t + z\,\gamma_w \quad \ldots\ldots (90b)$$

$$\sigma_t = \bar{\sigma}_{ft} + z\,\gamma_w \quad \ldots\ldots (90c)$$

in which $f(u_c)$ is the part of u_c transmitted to beneath the toe. The effective intergranular pressure is reduced below its initial value to

$$\bar{\sigma}_{ot} = \sigma_{ot} - f(u_c) \quad \ldots\ldots (91)$$

The shear strength is correspondingly reduced, and may be dangerously so, since an embankment induces high shear stresses in the zone beneath its toe.

By intercepting such laterally transmitted excess pressure, drain wells serve to minimize reduction in shear strength beneath an embankment toe. Although the danger from this lateral transmission of excess pressure becomes greater as the ratio of horizontal to vertical permeability increases, fortunately the same condition operates to improve the efficacy of the drain wells in attracting radial drainage.

Usage of Drain Wells.—Sand piles have been used to strengthen soft foundations for many years in the Low Countries of Europe and such sand columns undoubtedly accelerated consolidation when loads were added; although it is

questionable if constructors of the sand piles contemplated this added benefit or relied on it. To the writer's knowledge the first intended application as true drain wells occurred on the "SVIR 3" power dam constructed from 1930 to 1935 in Russia.[16] Here 12-in. sand-filled wells were installed 40 ft deep in a bed of interstratified clay and sand of Devonian age and appear to have served several purposes. Although they were initially installed for water control during excavation, they seem to have been used later for seepage control as relief wells for reducing seepage pressure in the more pervious buried strata. They also served to accelerate settlement of this concrete dam, as well as rate of gain of shear strength and corresponding resistance against sliding.

In American practice it appears that O. J. Porter, M. ASCE, and his associates at that time in the California Highway Department were the first to design and install sand-filled drain wells for the expressed purpose of accelerating consolidation—an experimental installation in 1934 followed by several highway embankments over marshes, peats, and harbor silts.[4,5] Early work was characterized by relatively large wells, up to about 30 in. in diameter. Subsequently, drain wells have been used considerably on the west coast of the United States on several important war construction and other projects. Recent large installations are on the Terminal Island Freeway near Los Angeles, Calif.,[17] and on the Bayshore Freeway near San Francisco, Calif.,[18] employing drain wells from 18 in. to 22 in. in diameter and from 30 ft to 60 ft deep. The objective of these installations has been largely for accelerating settlement and, to a lesser degree, for increasing shear strength to reduce toe displacements.

For similar purposes, the Connecticut Highway Department has employed drain wells of 12 in. in diameter and 25 ft in depth on 15-ft centers in crossing a marsh near Old Lyme, Conn.[19] An extensive installation is contemplated on a very wide or "dual-dual" highway near Elizabeth, N. J., using wells 18 in. in diameter on 10-ft centers up to 24 ft deep across a marsh.[20] It is particularly interesting that alternate bids on this New Jersey project showed a saving of more than 30% in favor of drain wells over excavation of the marsh deposit. An application almost solely for accelerating settlement is the experimental installation at LaGuardia Field, New York, N. Y.[21]—17-in. wells about 80 ft deep in silt with spacing varied from 8 ft to 14 ft.

Drain wells for the major purpose of increasing rate of gain in shear strength were employed during the construction of cellular cofferdam walls for sub-

[16] "Some Features in Connection with the Foundation of SVIR 3 Hydro-electric Power Development," by H. Graftio, *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., Harvard Univ., Cambridge, Mass., Vol. 1, 1936, pp. 284–290.

[4] "Studies of Fill Construction Over Mud Flats Including a Description of Experimental Construction Using Vertical Sand Drains to Hasten Stabilization," by O. J. Porter, *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., Harvard Univ., Cambridge, Mass., Vol. 1, 1936, pp. 229–235.

[5] "Studies of Fill Construction Over Mud Flats Including a Description of Experimental Construction Using Vertical Sand Drains to Hasten Stabilization," by O. J. Porter, *Proceedings*, Highway Research Board, Vol. 18, Pt. VI, pp. 129–141.

[17] "Vertical Sand Drains," *Roads and Streets*, August, 1946, p. 72.

[18] "Vertical Sand Drains Speed Consolidation of Soft Water-Bearing Subsoil," *Engineering News-Record*, March 6, 1947, p. 88.

[19] "Sand Drainage Wells Provide Possibilities for Economy," Highway Research Abstracts, Highway Research Board, October, 1947, p. 6.

[20] "Plan Use of Vertical Sand Drains Under New Jersey Superhighway," *Engineering News-Record*, October 23, 1947, p. 7.

[21] "Sand Drains for LaGuardia Field," *ibid.*, August 28, 1947, p. 7.

merged shipways.[6] Here 12-in. wells were placed inside the cofferdam cells to stiffen the filling of clay-like material formed from dredged-in marl. As an interesting innovation to speed further the consolidation of the cell fill, the central wells were pumped.

For the prime purpose of reducing lateral transmission of excess pressure, the first installation to the writer's knowledge was employed on the reconstruction of a slide on Riverfront Dike in Hartford, Conn.[22] As a matter of interest, this particular slide is one of the few that was analyzed and predicted in advance. A highway of dredged fill was being added landward soon after substantial completion of the dike, and the highway engineers decided to take the risk in the face of warnings on the probability of a slide, which occurred substantially as predicted. In designing the earthwork phases of the reconstruction for the Providence (R. I.) District of the Corps of Engineers, the writer and the author concentrated three lines of drain wells just inside the toe of the rebuilt embankment to insure against loss of shear strength at this critical zone.

Some ninety drain wells were constructed, in February and March, 1942, through 40 ft of sand and 40 ft of soft, varved clay (see Fig. 9). Wells were spaced in a triangular pattern on 20-ft centers and, to facilitate construction by hired labor equipment, were 6 in. in diameter. A 6-in. temporary casing was used through the sand, advancing the hole with a churn drill. To minimize smear, the clay was excavated by water jets of a special clay auger designed by F. E. Fahlquist, M. ASCE. Construction cost was about $1.00 per ft of well.

A group of closed-system piezometers was in place, with which it was hoped to measure the effect of these drain wells. However, the effect of daily temperature variations from darkness to daylight made the closed-system piezometers very unsatisfactory and, by removing the Bourdon gages, they were converted to the open-system type (open pipe with filter at the tip). After this change the piezometers also failed to give dependable readings because of the effect of dampening on changing pressures, from the flow into an open-system piezometer from a highly impervious soil. Accordingly, no dependable measurements were obtained on the performance of these drain wells except that, from general observations, it appeared satisfactory.

One question that arose in the design of these Riverfront Dike drain wells concerned a possible tendency of the well to act as a sand pile or hard point in the foundation, attracting load and somewhat relieving stresses in the surrounding compressible deposit. Although this tendency would be less for wells of small diameter than for larger wells with a smaller slenderness ratio, it was decided to place two cushion layers of cinders on the Riverfront Dike wells, with the thought that this material would be more compressible than the sand backfill, and substantially as pervious. However, the cinders obtainable proved to be a poor filter against intrusion of the silt strata in the surrounding varved clay and were difficult to install, tending to sink through water slowly and to mix with the sand. Accordingly, use of the cinder cushion layers was

[6] "Submerged Shipways with Steel Sheeting Walls," by Adolph J. Ackerman and C. B. Jansen, *Civil Engineering*, July, 1943, p. 309.

[22] "Foundation Failure Causes Slump in Big Dike at Hartford, Conn.," *Engineering News-Record*, July 31, 1941, p. 142.

abandoned and most of the wells were backfilled entirely with sand, poured in from the surface.

If a drain well should act as a stiff point in attracting load in spite of the loose state of its sand backfill, the effect on arching of load would apparently be the opposite of the author's solution for equal vertical strain. The effect on consolidation rate would probably not be large, judging from the small difference in rates for free strain and for equal vertical strain shown in Fig. 8. Of more concern is the action of the sand column in deforming under a load concentration as the surrounding, and probably more compressible, soil settles. This deformation may be accomplished either by a bulging of the well column of sand with a small increase in density, or by a local shearing with possibly a relative displacement of the sand column. Bulging is not objectionable and is believed to be the more probable action in soft foundation soils where most drain wells have been installed to date, except on the SVIR 3 dam.[16] For example, to accommodate about 4 ft of settlement, an 18-in. drain well 80 ft long would have to increase uniformly in diameter only about 0.4 in., and considerably less than this by allowing for the densification of the sand backfill from its initially loose state. A definite shearing of the well column could be very undesirable because, if continuity of the drainage path were broken, the break might even nullify the effect on the consolidation contributed by the region of the well below the blocked zone. Such shearing of the sand column would seem more likely to occur in brittle, but still quite compressible, clays and, pending further knowledge, is probably best handled by conservatism in selecting well spacing and size.

In what is believed to be the first application of drain wells to the design of an earth dam, from 1943 to 1945, the writer and the author designed a system of about 240 drain wells to be placed in a triangular pattern on 60-ft centers over substantially the entire valley bottom of Claremont Dam, near Claremont, N. H. Although the design of this dam was essentially completed in 1945 by the Providence District, Corps of Engineers, its construction has been deferred pending further negotiation with local interests. The dam is intended to be about 120 ft high over a foundation of approximately 15 ft of sand and gravel lying over from 40 ft to 80 ft of soft, varved silt (silt, clay, and fine sand interstratified, as shown in Fig. 9, with silt predominant). The weak silt is underlain by impervious glacial till which is likely to create a case of single drainage where excess pore water can escape only in an upward direction. Drain wells 18 in. in diameter, extended to the bottom of the silt with methods to minimize smear, were included in the design because flat slopes to reduce shear stresses and an extended construction period of gradual load application were not in themselves considered sufficient for stability with this very weak silt foundation. The wells are intended both to reduce lateral transmission of excess pressure and to accelerate rate of gain of shear strength.

Conclusions.—For the design of a drain well installation, the author's solutions are excellent examples of the case where a mathematical solution is invaluable in indicating the relative importance of the different variables—ratio of horizontal to vertical permeability, well spacing, well resistance, peripheral smear, and well size. In general, these can be evaluated from the curves which

the author has laboriously computed and presented in this paper and in his earlier paper.[7] If the author has other similar curves available, it is felt they would be a worthwhile inclusion in the closing discussion, particularly where they would aid in showing the effect of single variables and comparing the relative importance of the different variables.

The author has properly indicated that certain solutions were developed by L. Rendulic[8] in 1934 and by N. Carillo[9,10] in 1940.[23] For the record, it is worthy of mention that the author also developed substantially the same solutions independently from 1940 to 1942 before learning of the solutions of Messrs. Rendulic and Carillo.

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[7] "The Influence of Drain Walls on the Consolidation of Fine-Grained Soils," by R. A. Barron, Providence (R. I.) Dist., U. S. Engr. Office, 1944.

[8] "Der Hydrodynamische Spannungsausgleich in Zentral Entwasserten Tonzylindern," by L. Rendulic, *Wasserwirtsch, u, Technik*, Vol. 2, Jahigang, 1935, pp. 250–253 and pp. 269–273.

[9] "Consolidation of a Soil Stratum Drained by Wells," by N. Carrillo, Harvard Univ., Cambridge, Mass., 1941 (mimeographed).

[10] "Simple Two and Three Dimensional Cases in the Theory of Consolidation of Soils," by N. Carrillo, *Journal of Mathematics and Physics*, March, 1942.

[23] "Drainage of Clay Strata by Filter Wells," by K. Terzaghi, *Civil Engineering*, October, 1945, p. 463.